A PRACTICAL TREATISE

ON

SUB-AQUEOUS FOUNDATIONS

INCLUDING

THE COFFER-DAM PROCESS FOR PIERS

AND

DREDGES AND DREDGING

BOOKS BY THE SAME AUTHOR

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[Frontispiece.]

THE PONT DU GARD, NIMES, FRANCE.

A PRACTICAL TREATISE ON

SUB-AQUEOUS FOUNDATIONS

INCLUDING

THE COFFER-DAM PROCESS FOR PIERS

AND DREDGES AND DREDGING

WITH

NUMEROUS PRACTICAL EXAMPLES FROM ACTUAL WORK

BY CHARLES EVAN FOWLER
CONSULTING CIVIL ENGINEER
Member American Society of Civil Engineers; Member Canadian Society of Civil Engineers; Past President Pacific Northwest Society of Engineers; Past President Seattle Board of Park Commissioners; Special Lecturer University of Washington

"Much of the success of any one in any kind of work, and especially in work subject to the peculiar difficulties of that we are considering, depends upon the spirit in which it is undertaken."—ARTHUR MELLEN WELLINGTON.

THIRD EDITION, REVISED AND ENLARGED

SECOND THOUSAND

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THIS WORK IS DEDICATED TO

Professor George C. Comstock

Director of Washburn Observatory

AND TO THE MEMORY OF

Professor C. N. Brown

Late Professor of Civil Engineering, Ohio State University

BOTH OF WHOM INCULCATED AN ABIDING DESIRE FOR THE APPLICATION OF MATHEMATICS AND THE SCIENCES TO THE PRACTICAL SIDE OF ENGINEERING

PREFACE TO THIRD EDITION

THE necessity for revising the text for a new edition of "Ordinary Foundations" caused a general review to be made of the literature on this subject and disclosed the fact that there was need for a more extensive treatise than had yet appeared on this subject. The available material from which to draw for the new matter is so very extensive and the immense amount of data accumulated by the author in a varied practice of over twenty-six years, has made it difficult to decide just what limits to set for a "Treatise on Subaqueous Foundations."

The new matter is largely practical examples of work, much of which has been constructed by the author or with which he has come in intimate contact in a consulting capacity, and the working plans of many different kinds of plant have been included.

These have had the test of actual construction and use, and while they are not always perfect, the fact of actual use having proved their worth, makes them of peculiar value to the engineer not having other data of similar kind at hand.

The chapter on pile-driving has been enlarged by giving conclusions arrived at on construction work as to the methods best employed on work of this character. The use of concrete piles has become of such wide application that data on their construction and driving has been added.

The jetting of piles has become so extensive as to demand a whole chapter for giving the details of such plant and for a full discussion of the best methods to employ. But little has been recorded as to the experience of others, and the methods used by the author can be easily and intelligently modified for any particular case.

The growing use of metal sheet-piling has made it necessary to add considerable data on this subject. Where timber has become scarce and expensive they must be used, and in locations where

timber is cheap, they can be economically employed, by using them over again on other contracts.

The making of foundations by the open-dredging process has been further treated by describing in much detail two additional pieces of work, of considerable difficulty, carried out by the author on some recent construction.

Additional data on the use of compressed-air in caisson work have been given in the description of the great steel caissons of the Forth Bridge, in the account of the steel caissons of the Antwerp quay wall, and in a detailed description of the timber caissons of the Columbia River bridge at Vancouver, Wash. The subject of caisson disease has been summarized, and the subject of decompression of the workmen discussed fully enough for all practical purposes.

The elementary ideas of pneumatic caisson work may best be obtained from the chapter on Divers and Diving, which will also prove valuable for reference to the engineer on any kind of sub-aqueous work, the subject having been treated fully enough to make it unnecessary to refer elsewhere for information on this subject.

The subject of removing old piers has been concentrated in a chapter on that subject. A more extended discussion of dredges and dredging has been deemed necessary for a comprehensive knowledge by the engineer of sub-aqueous work, so that three chapters on this subject have been added.

Practically all foundation work requires the use of launches, tugs and scows, and as there is practically no literature on this subject, an entire chapter has been devoted to it, giving full enough data and enough plans and working drawings to enable the engineer to act intelligently in buying, constructing and operating floating plant. It is hoped the author's experience of many years' extensive use of tugs, scows, floating drivers, derrick scows and dredges will enable others to start on a better basis than is the case where the inland engineer finds himself suddenly dropped down on salt-water work.

Considerable data has been added on the bearing power of soils, including Corthell's valuable conclusions. The data on friction on piles and caissons has been fully covered and the index will disclose many references to it, including the most valuable information of this kind.

The subject of quarrying riprap rock for use in foundations has been given a chapter, as the engineer will from this source gain valuable data as well on the use of explosives and on rock work in general.

Calculation of piers, retaining walls and footings has been added

simply in the desire to fully cover the subject of foundations in this volume. To the chapter on cement and concrete have been added by permission the valuable tables of Taylor and Thompson and such other data as was lacking in former editions. Forms for concrete have been given in a separate chapter, to such an extent as will give a basis for any class of form work.

Entirely new matter has been given on Piers and Wharves; Dams, Seawalls and Retaining Walls; Dry Docks and Locks; and two chapters on the Cost of Foundation Work, in the endeavor to so cover the subject of Foundations that the Engineer will not have to go to any other source to properly execute such work. The many new tables and the novel treatment of many subjects will, it is hoped, meet with the approval of the practicing engineer.

Acknowledgment is due particularly to T. G. McCrory, Asso. Mem. Am. Soc. C.E., for assistance in preparing many of the drawings and in helping to assemble the data; to H. D. Fowler for many of the photos used in illustrating various subjects, and for drawings prepared; to Taylor and Thompson for permission to use tables from "Concrete Plain and Reinforced"; to A. H. Fuller, M. Am. Soc. C.E., Professor of Civil Engineering, and to C. C. More, Asso. M. Am. Soc. C.E., Professor of Mechanics, at the University of Washington, for valuable consultation; and to many others to whom acknowledgment has been made in the text.

C. E. F.

SEATTLE, WASH.,
December, 1913.

PREFACE TO SECOND EDITION

THE necessity for a second edition of this book has made it possible to make valuable additions to the text, by which the subject of ordinary foundations is more comprehensively covered. The construction of piers by the use of metal cylinders; with timber caissons by open dredging; and the construction of ordinary-sized foundations by the use of pneumatic caissons, have furnished another chapter. A chapter has been added on the subject of foundations, which covers the bearing capacity of soils.

The new chapter on building stone, masonry, and the design of piers is intended to supplement the old chapter on the "Location and Design of Piers."

Enough new matter has been added on cement and concrete to form an additional chapter, which includes valuable tables giving the amount of material required for concrete of different proportions.

The building of piers of timber and pile bents, together with the subject of timber preservation, has been discussed in a final chapter, as fully as a general knowledge requires.

It is hoped that in enlarging the field covered by "The Cofferdam Process for Piers," the book will be more valuable for the practicing engineer, besides giving a general enough treatment to make it valuable for class work.

C. E. F.

SEATTLE, 1904.

INTRODUCTION

The greater part of foundation work is of an ordinary character. And while difficult foundations have been quite fully treated by engineering writers, ordinary ones have too often been passed over with mere mention, or treated in such a general way that the information proves of little value in actual practice.

Many valuable examples of work of this character have been described in current engineering literature, and it is hoped that by bringing them together a real service will be rendered the profession, as well as much valuable time be saved for considering other and equally important problems.

The history of the coffer-dam process would seem to indicate that engineers of nearly a century ago gave more consideration to the smaller problems than the engineer of to-day, who has apparently passed to the consideration of the larger and of course more interesting ones.

That this is deplorable, is proven by the many cases where money has been wasted in the after effort to make good the mistakes that have become apparent where cheap construction of coffer-dams has been resorted to. The saving in original cost, as between an indefensible method and a defensible one, is often so small as to seem absurd when it has become necessary to make large expenditures to rectify the errors.

Errors of judgment are more easily excusable with regard to foundations than with any other class of construction, but where definite limits can be set, economy will result by keeping as closely as possible within them.

Reference is made in the following pages to the splendid construction of foundations by the Romans, where they could be built outside the water. The Pont du Gard, illustrated in the frontispiece, is the most notable example of this extant. It is interesting also as indicating their knowledge of the better form of piers and methods of arch construction.

Although constructed during the reign of the Emperor Augustus, at the beginning of the Christian era, it is in a remarkable state of preservation, aside from repairs that have been made from time to time.

Probably the earliest recorded examples of the use of coffer-dams which give details of construction are those constructed under the engineers of the Ponts et Chausées.

Those built under Perronet at the bridge of Orleans were large and extensive, and references made to the pile-drivers and the pumps used on the work, serve to illustrate the great amount of attention paid to planning the details of construction.

The same engineer completed the piers of the bridge at Mantes, where the coffer-dams were constructed to inclose both the abutment and the nearest pier within one dam, making the dimensions about 150 feet by 200 feet in the extreme.

Hardly less notable were the coffer-dams at Neuilly, where the interiors were so large that the excavation did not approach near the inside wall of the dam.

All of these were constructed prior to the year 1775, and the details as shown in the elaborate drawings are of much interest to the engineer engaged on similar works.

The coffer-dams constructed about 1825 by Rennie on the new London bridge were the prototypes of those used at Buda Pesth, but were elliptical in form. They were designed with as much care, apparently, as any other feature of the bridge, and from the fact that the water was pumped to twenty-nine feet below low water and the work found tight, the details must have been very carefully executed.

However great the amount of care bestowed, there will be cases undoubtedly where the difficulties cannot be foreseen, and it will become necessary to adopt some of the many expedients cited to overcome them; or they might better be employed from the start, where any suspicion is had that trouble may ensue.

The question as to whether it will be best to use a crib or a sheet-pile coffer-dam will almost always be decided by the character of the bottom, the location, and the character of the foundation to be built. It is advisable, whichever type is selected, to make the size large enough, so that the excavation may be completed without approaching too close to the inside wall of the dam, and so that plenty of room may be had for the laying of the foundation-courses.

The unit stress adopted for timber construction is believed to be as large as will give good results in the majority of cases, both on

account of the possibility of the construction having to undergo more severe usage than is expected, and on account of the grade of timber which is most often made use of for temporary works.

Where it is permissible from the standpoint of true economy, it is believed that steel construction will commend itself for use. In most localities it will not be long until metal construction will be found cheaper than timber for building coffer-dams, and in many places this is already true.

A great mistake is made, in nearly nine cases out of ten, by trying to use old machinery, such as hoisting-engines, pumps, and the like, which are ill adapted to the purposes for which they are intended, on account of lack of capacity, and only too often on account of having outgrown their usefulness.

The engineer would avoid many unpleasant situations by demanding that a proper outfit be provided, and in the end gain the thanks of the contractor for increased profits.

Extended acquaintance with Portland cement is increasing the use of concrete in construction, and this is a great gain for the engineer, as it is not only superior to much stone that is used, but is better adapted to use in difficult situations. It also lends itself more readily to use for ornamental details in pier construction. That truly ornamental piers are not, however, those with needless and frivolous details, has been clearly set forth in the last article. Simplicity and beauty are near relatives. The best locations cannot always be chosen for piers, but careful examination will often be the means by which bad locations may be avoided.

The methods for determining the economic division of a given crossing of a river have not come into general use, probably on account of lack of easy application. The method given is an accurate one and very simple to use, especially if the results are tabulated for a given loading.

C. E. F.

NEW YORK CITY, 1900.

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TABLE OF COFFER-DAMS.—

No.	Page.	River and Location.	Current.	Water Head.	Character of Bottom.
1	6	River 200 feet wide, Ohio..........	None.	12′+	Cemented gravel.
2	6	Clyde at Glasgow................	Slight.	9′+	Gravel, sand, mud.
3	8	Estuary or Harbor................	Tide.	40′	Sand & gravel over clay.
4	9	Danube at Buda Pesth............	Swift.	54′±	Gravel over clay.
5	23	Kanawha near mouth.............	Swift.	34′−	Gravel over hardpan.
6	25	Ohio near head..................	Moderate.	20′+	Gravel.
7	25	Western part United States........	Moderate.	6′+	Soft.
8	26	St. Lawrence, lower river..........	Swift.	20′+	Rock.
9	27	Arnprior Bridge..................	Swift.	21′+	Rock.
10	29	New Mexico, underflow...........	None.	15′+	Sand.
11	29	Arkansas at Tulsa................	Moderate.	7′+	Gravel over rock.
12	29	Western part United States........	Moderate.	6′+	Soft.
13	29	Republican in Kansas.............	Moderate.	6′+	Sandy.
14	29	Western part United States........	Moderate.	7′+	Gravel over soapstone.
15	29	Western part United States........	Moderate.	6′+	Rock.
16	29	Payette and Weiser, Union Pac....	Moderate.	6′	Soft.
17	31	Mississippi, Fort Madison.........	Swift.	19′	Soft.
18	31	Schuylkill near Phila., Pa.........	Moderate.	Deep.	Mud over rock.
19	36	Green River, Wash...............	Swift.	18′	Gravel and boulders.
20	40	U. S. Canal, Keokuk..............	None.	12′+	Rock.
21	45	Mississippi, St. Louis.............	Swift.	22′	Rock.
22	46	Queen's Bridge..................	Swift.	15′	Rock.
23	47	Harlem Ship Canal...............	Moderate.	25′	Rock.
24	49	Arthur Kill Bridge................	Tide.	28′	Clay over rock.
25	50	Coteau Bridge, C. Pac. Ry........	Moderate.	28′	Rock.
26	50	Mystic River, Boston.............	Tide.	38′	Rock.
27	115	Ann Arbor, Mich., M. C. Ry.	Moderate.	6′+	Gravel.
28	116	Arthur Kill Bridge................	Tide.	30′−	Mud and clay.
29	116	Boston Harbor, sewer.............	Tide.	10′	Sand and gravel.
30	119	Illinois River, La Grange..........	Moderate.	7′	Sand and mud.
31	119	Kankakee at Momence............	Moderate.	6′+	Rock.
32	120	Potomac at Harper's Ferry........	Swift.	6′+	Rock.
33	120	Charlestown Bridge, Boston.......	Tide.	6′+	Soft.
34	121	Fort Monroe, sewer...............	None.	20′	Soft.
35	123	Seattle, Wash....................	Tide.....	32′+	Soft sand.
36	129	Tennessee at Chattanooga.........	Swift.	8′+	Gravel over rock.
37	130	Cumberland, Md.................	Moderate.	10′+	Sand over hardpan.
38	132	Mississippi, Sandy Lake...........	Swift.	8′+	Sand.
39	134	Arkansas, Little Rock.............	Moderate.	6′+	Sand.
40	271	Farnitz, Stettin, Germany.........	Moderate.	25′+	Clay.
41	273	Coosa, Gadsden, Ala..............	Moderate.	10′+	Gravel over rock.
42	134	Schuylkill, P. & R. R. R..........	Swift.	8′+	Rock.
43	137	St. Helier Bridge, Jersey, Eng......	Tide.	13′+	Earth over rock.
44	137	Thames at Putney................	Moderate.	Deep.	Mud.
45	137	Victoria (B. C.) Docks...........	Tide.	35′	Rock.
46	138	Kaw at Topeka..................	Swift.	6′+	Sand.
47	162	Firth of Forth..................	Tide.	15′+	Rock.

SYNOPSIS OF EXAMPLES

Form of Construction.	Inside Dimensions.	Kind of Puddle.	Thickness Puddle.	Remarks.	Page.	No.
Earth bank.	10′×60′?	Clay and gravel.	5′+	No leaks.	6	1
Sheet-piles.	20′×58′?	Clay.	3′		6	2
Sheet-piles.	Large.	Clay, sand, & gravel.	3–6′	Typical.	8	3
Sheet-piles.	72′×136′+	Clay and gravel.	2–5′	Difficult.	9	4
Earth bank.	90′×330′	Clay and gravel.	19′+		23	5
Earth bank.?	200′×600′	Clay and gravel.		Failed.	25	6
Crib.	Medium.	Clay.	3′+		25	7
Crib, single.	24′×43′	Concrete inside.			26	8
Crib, single.	16′×34′	Concrete inside.			27	9
Crib, single.	17′×43′	Clay outside.		Special.	29	10
Crib, single.	Medium.	Clay outside.			29	11
Sheet-piles.	Medium.			Typical.	29	12
Sheet-piles.	Medium.	Clay outside.			29	13
Sheet-piles.	Medium.	Clay outside.			29	14
Sheet-piles.	Medium.	Clay.	Equal depth.		29	15
Box or crib.	12′×36′	None.		On grillage.	29	16
Staves.	36′ diam.	None.		On grillage.	31	17
Sheet-piles.	80′ diam.	None.		Failed.	31	18
Log crib.	Large.	Clay and gravel.	6′+	Seep. large.	36	19
Canvas on plank.	80′ long.	Rotten manure.		Bulkhead.	40	20
Crib, double.	28′×64′	Clay.	3′ 0″	Canvas used.	45	21
Box and canvas.	Square.	Clay oustide.		Movable.	46	22
Polygon crib.	47′ diam.	Clay.	4′ 6″		47	23
Polygon crib.	44′ diam.	Clay and gravel.	5′ 0″		49	24
Crib, single.	34′ diam.	Concrete inside.			50	25
Basket crib.	60′ diam.	None.			50	26
Sheet-piles.	13′×44′	Clay and gravel.	2′ 8″		115	27
Sheet-piles.	Large.	None.		Two trials.	116	28
Sheet-piles.	12′ wide.	Clay.	6–8′		116	29
Sheet-piles.	Medium.	None.			119	30
Sheet-piles.	Medium.	Gravel.		Two trials.	119	31
Sheet-piles.	Medium.	Gravelly clay.			120	32
Sheet-piles.	18′6″×119′	Concrete inside.			120	33
Sheet-piles.	44′ diam.	Sand and concrete.	7′+		121	34
Sheet-piles.	30′×56′	None.		Very Tight.	123	35
Sheet-piles.	Large.	Clay.	9′ 0″		129	36
Sheet-piles.	15′×50′	None.			130	37
Sheet-piles.	829′ long.	Clay.	8′±		132	38
Sheet-piles.	16′×38′	Earth outside.			134	39
Sheet-piles.	23′×55′±	Clay.	2–4′	Removal.	271	40
Sheet-piles.	28′×28′±	Clay.	12′+	Removal.	273	41
Sheet-piles.	16′×42′.	Clay and gravel.	8′+	Movable.	134	42
Sheet-piles.	Medium.	Clay outside.			137	43
Sheet-piles.	Medium.	None.			137	44
Sheet-piles.	500′ long.	Clay.	2–7′		137	45
Sheet-piles.	18′×55′	Clay outside.			138	46
Metal.	60′ diam.	Concrete seal.			162	47

SUB-AQUEOUS FOUNDATIONS

CHAPTER I

HISTORICAL DEVELOPMENT

THE continued increase in the weight of our bridge superstructures and of the loads they have to carry has led to increased care, to a very gratifying degree, in the preparation of the foundations for bridge piers and abutments.

An old authority very truly states "The most refined elegance of taste as applied in the architectural embellishment of the structure; the most scientific arrangement of the spans and disposition generally of the superior parts of the work; and the most judicious and workmanlike selection and subsequent combination of the whole materials composing the edifice, are evidently secondary to the grand object of producing certain firm and solid bases whereon to carry up to any required height the various pedestals of support for the spans of the bridge."

There is every reason to believe, from the bridges of the Romans still extant and of those of ancient and mediæval times of which there are remains or records, that the foundations were carefully considered.

The most ancient form was likely begun by dumping in loose stones until the surface of the water was reached and the masonry could then be commenced without the necessity for any method of excluding the water. The oldest civilizations were in tropical or semi-tropical countries where the streams are dry beds for many months in the year and suitable foundations were easily made without water to contend with. Where the bottom of the stream was rock, the engineering could be very little improved upon to-day, and even where there was shallow water on rock bottom, the piers were well founded in the shallowest places; the bridge often winding

across the stream in serpentine form, similar to the bridge over the river Karun, at Shuster, Persia. (Fig. 1.)

The arch was developed to such an extent by the Romans and the spans were increased to a length which rendered the construction of piers in the water unnecessary for short bridges, the abutments or skew-backs being without the stream on either bank.

The difficulty of founding piers in midstream was doubtless the controlling cause for the larger spans, such as the one built at Trezzo, over the river Adda, by order of the Duke of Milan, some time prior to the year 1390. The span at low water was 251 feet, the single arch being of granite in two courses. The placing of a middle support was doubtless found to be impracticable and caused the design of an arch which has seldom been equaled or eclipsed. (Fig. 2.)

The construction of roads has ever been the harbinger of civilization, and with the spread of civilization came a demand for the improvement of means of communication. The engineer was called upon to construct better and greater bridges in a permanent manner, which led to the origin and development of the four methods for founding in water that were used in olden times. These may be classified as, first, the method with open caissons; second, the use of piles with a capping of coarse concrete about the tops; third, the use of piles after the manner of the French encaissement; and fourth, the use of coffer-dams. A fifth method might be added, in which the bridge was built on dry land adjacent to the stream, and the river diverted to a new channel afterwards excavated under the completed structure. This is, however, an avoidance rather than a solution, unless the river is to be diverted in the course of its improvement.

The first method, as described in old treatises or accounts, consisted of little more than baskets formed of branches of trees, weighted with stone to sink them, and after sinking filled with loose stone to near low-water level, where the masonry could be commenced. These baskets were similar in construction to the mattresses used in the bank revetment of the Mississippi or the bamboo casings used by the Japanese to hold stones in place on bank protection.

An improvement was effected by using in place of baskets, boxes or small open caissons which were sunk and filled in the same manner, several being used for one pier. This was the method used at Blackfriars bridge and also at Westminster bridge, over the Thames, and has been much used in recent times, the caisson

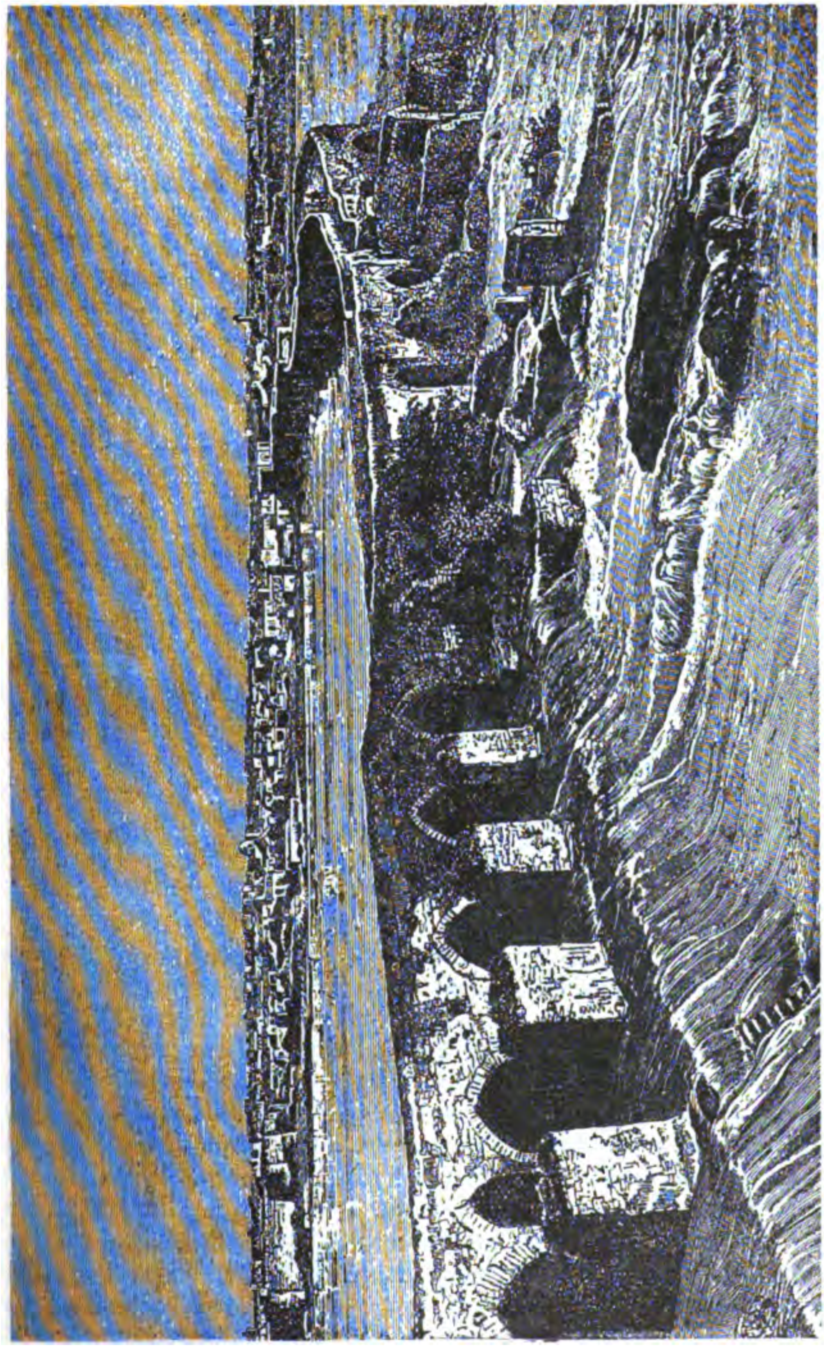


FIG. 1.—BRIDGE AT SHUSTER, PERSIA, OVER THE RIVER KARUN.

being built large and strong enough for the entire pier, which is built up as the caisson sinks.

The second method consisted of driving piles over the area of the foundation until the heads were below low-water level, and spaced at distances apart as required by the nature of the bottom, similar to the methods in vogue to-day. The heads of the piles were not driven to the same level, however, and were incased in a form of coarse concrete such as was used by the Romans, but what is now called beton. This was leveled up and on it was laid the stone for the footing course of the pier.

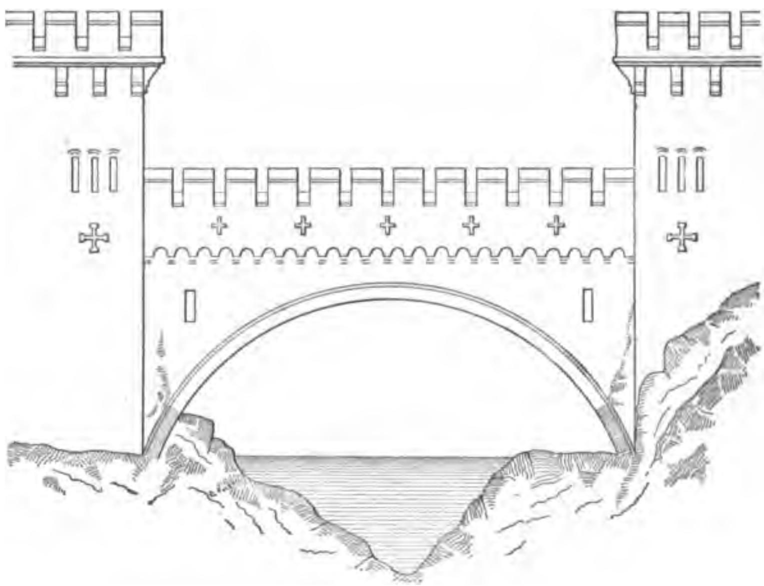


FIG. 2.—BRIDGE OVER THE ADDA AT TREZZO, MILAN, A PROBABLE RESTORATION.
(The shaded portion of arch rings is all that remains.)

The third method of encaissement was probably an improvement of the dumping in of loose stone on which to place the pier, and consisted in inclosing the space for the pier with sheet-piling, after which the loose material was removed from the bottom as much as possible and the stone dumped inside until nearly up to low water, at which time the pier could be begun.

These last two methods doubtless met with much favor, owing to the familiarity with pile-driving, in which the Romans especially were proficient. Cæsar's bridge over the Rhine was built entirely on piles, and in a view of it after the old print in the Museum de St.

Germaine, is pictured a pile-driver on a float in position for driving. (Fig. 3.)

This third method was the early type of the crib, which has been such a factor in the building of the earlier foundations over our American rivers. Crossed timbers laid up crib fashion with rectangular openings or cells between the timbers were sunk and filled with broken stone on which to build the pier.

These methods were all deficient in affording no means of seeing or making a careful examination of the bottom on which the foundation was to be placed, and with the advent of more permanent structures of greater magnitude the coffer-dam came into use. This

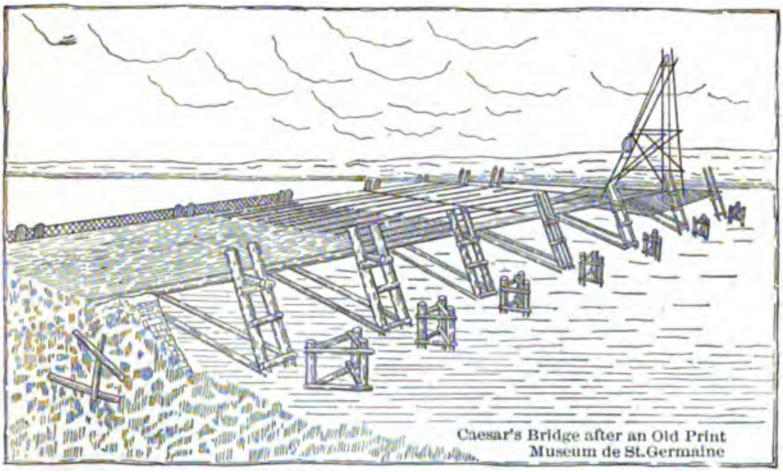


FIG. 3.—CÆSAR'S BRIDGE OVER THE RHINE.

allowed the bottom to be freed from water, and after a careful examination and preparation of the foundation, the work could proceed in the dry until above water-level.

The pneumatic caisson is now in general use for all foundations that must go to any considerable depth below the water and has even been used in some instances where the depth was slight, but where for various reasons it was deemed expedient to use compressed-air caissons. Recent expressions from some engineers of high standing would indicate that they do not consider it good practice to use coffer-dams in any case, one making the statement that he had not used a coffer-dam for thirty years, while another seemed to think it a matter to be left to the pleasure of the contractor. That the use of this method has gotten into disfavor seems to be

beyond question and it will be the purpose of the succeeding pages to learn to some extent why this is so, but mainly to show from successful examples how to proceed, that success instead of failure may result. Any attempt to account for the origin of the coffer-dam process would be futile, inasmuch as the savage, wishing to free a space from water, doubtless banked up earth about the area and, scooping out the water with his hands, laid the ground bare for inspection. From so simple a beginning, the first method likely to occur to a mind capable of reasoning, can readily be imagined the course of development of coffer-dams.

The most simple form in use at the present time, where the water is quiet, is shown in Fig. 4, and consists principally of a bank of earth which is made thick enough to be nearly or quite impervious to water, the earth being prevented from caving into the excavation by piles supporting a timber casing. Some of the recorded examples of the early use of this process are of interest in illustrating the care which has been bestowed upon their construction in important works and will call attention to that incessant care which is necessary to success in any work of this character.

Robert Stevenson, the great English engineer, thought it not beneath his dignity to give full instructions as to the construction of the coffer-dams for the Hutcheson bridge over the Clyde at Glasgow. The bridge consisted of five arch spans, the total length between the abutments being 404 feet and the width 38 feet. The four piers were from 11 to 12 feet in thickness, being designed to take up the arch-thrust, and 48 feet in length at the footing. The specifications written at Edinburgh in April, 1828, are so explicit that they will be quoted in full on this point: "It having been ascertained by boring and mining that the subsoils of the bed of the river consist of gravel, sand, and mud to the depth of 27 feet and upwards, it becomes necessary to prepare foundations of pile-work for the bridge; and, therefore, to insure the proper and safe execution of the works, coffer-dams are to be constructed around each of the foundation-pits of the two abutments and four piers of such dimensions as to afford ample space for driving piles, fixing wale-pieces, laying platforms, pumping water, and setting the masonry; and likewise for the construction of an inner or double coffer-dam should this ultimately be found necessary. The framework of the coffer-dams is to consist of not less than two rows of standard or gage- and sheeting-piles, kept at not less than 3 feet apart for the thickness of a puddle-wall or dyke, which space is to be dredged to a depth of not less than 9 feet under the level of the summer water-mark above de-

scribed, before the sheeting-piles are driven. The gage or standard piles are to measure not less than 24 feet in length and 10 inches square. They are to be placed 3 yards apart and driven perpendicularly into the bed of the river to the depth of 16 feet under the level of the summer water-mark, thereby leaving 8 feet of

FIG 4.—A PRIMITIVE SOLUTION.

their length above that mark. Runners or wale-pieces of timber 9 inches square are then to be fitted on both sides of each row of gage-piles, to which they are to be fixed with two screw-bolts of not less than 1 inch in diameter, passing through each of the gage-piles. One set of these inside and outside wale-pieces is to be placed at or below the level of summer water-mark, and the other

set within 1 foot of the top of each row of said piles, the whole to be fixed with screw-bolts in the manner above described. The wale-pieces are to be $4\frac{1}{2}$ inches apart in order to receive and guide the sheeting-piles. This is to be effected by notching the wale-pieces into the gage-piles. The sheeting-piles are to be 21 feet in length, $4\frac{1}{2}$ inches in thickness, and not exceeding 9 inches in breadth. They are to be closely driven, edge to edge, along the space left between the walings, and each compartment of the sheeting between the gage-piles is to be tightened with a key-pile. The coffer-dam frames are to be properly connected with stretchers and braces before commencing the interior excavation. Each coffer-dam is to be provided with a draw-sluice, 14 inches square in the void, with a corresponding conduit passing through the puddle-dyke at the level of summer water-mark. To render the coffer-dams water-tight the whole excavated space between the two rows of piling is to be carefully cleared of gravel, sand, or other matters, to the specified depth, and clay well punned or puddled is then to be filled in and carried up to the level of the top of the sheeting-piles. But if it shall, notwithstanding, be found that the single tiers of coffer-dam do not keep the foundation-pits sufficiently free of water for building operations, the water must either be pumped out and kept perfectly under by steam or other power, or else excluded by the construction of a second tier of coffer-dam similar in construction to the first. For the foundation-pits of the two abutment piers on either side of the river it is not expected that more will be required on the landward side for keeping up the stuff than a single row of gage- and sheeting-piles; but if the engineer shall find other works necessary upon opening the ground they must be executed by the contractor and shall be paid for agreeably to the contract schedule of prices for the regulation of extra and short works. The stuff within the coffer-dams is to be excavated to the depth of 10 feet under the level of summer water-mark for each of the piers and 8 feet for each of the abutments."

The present practice of leaving all this to a contractor, whose idea is too often to sacrifice everything to cheapness, appears in very unfavorable contrast to this careful description.

An article on founding by means of coffer-dams, published in 1843, gives directions for placing a coffer-dam in 40 feet of tide-water; and although the engineer of to-day might use some other method for such a depth, an illustration (Fig. 5) and short description of it are given, as ideas may be gained for application to ordinary works.

The water was assumed at 10 feet deep for low tide, 28 feet at high tide, with 12 feet of sand and gravel to be removed to expose the clay on which the pier was to rest. Four rows of piles were to be driven around the area, the outer row to within 1 foot of low water, the two rows in the middle to within 3 feet of high water, the inner row to 11 feet above low water, and all to be down 5 feet into the clay. The outer row of piles to be 6″×12″, the two rows in the middle 12″×12″, and the inner row 8″×12″; all driven close together and to have waling-pieces, braces, and brace rods as shown in cross-section. The rows to be 6 feet apart and to be filled in between with a puddle of clay mixed with sand and gravel.

The report of W. Tierney Clark, the engineer of the Buda Pesth suspension bridge, gives an account of what are probably the largest

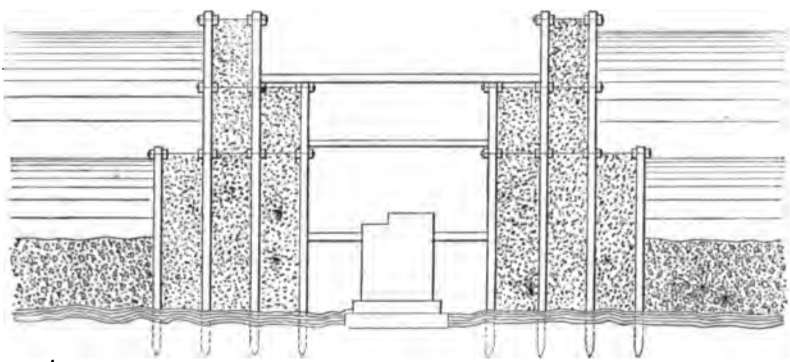


FIG. 5.—COFFER-DAM IN TIDE-WATER.

bridge coffer-dams ever constructed. Some other method would now be used for such a location, but this fact will not detract from the lessons that may be drawn from them.

The Danube was crossed at Buda Pesth previous to the year 1837 by means of a bridge of boats which had to be taken up during the winter and the passage made by ferry or on the ice, so that for six months of each year there was great risk in crossing and frequent loss of life. The building of a permanent bridge was brought about through the efforts of the Count Szechenyi, who, as a member of a committee, proceeded to England in 1832 and after a careful investigation of existing works decided upon the construction of a suspension bridge. The greatest question for solution was the founding of the two towers in a river like the Danube, where the ice throughout the long winter wrought havoc with everything in reach. The ice in the river in February, 1838, was from 6 to 10 feet thick

near the site of the proposed piers. On March 9, a movement occurred across the whole river and for a length of 350 yards, the whole moving in a solid mass. On March 13 it moved again 400 yards and three hours later a general breaking began. The ice piled up on the shoals, causing a sudden rise to 29 feet 5 inches above zero, and while it was at this height for only a few hours, it is recorded that a great part of Buda and two-thirds of Pesth were destroyed and many lives lost.

The extraordinary design of the coffer-dams can the more readily be understood after this description, it being doubted by many

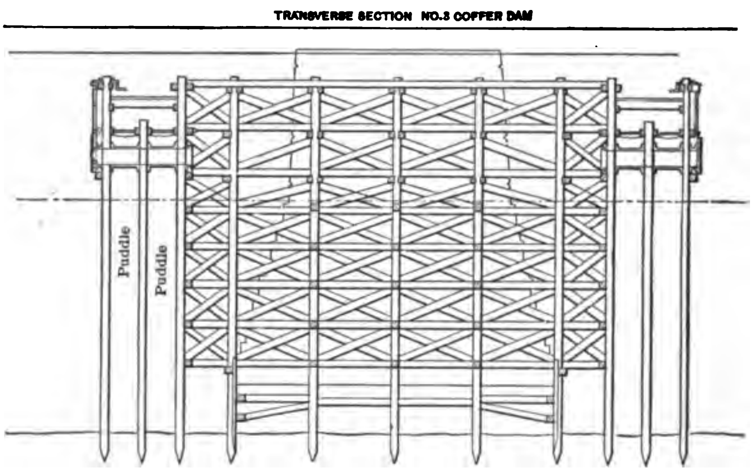


FIG. 6.—BUDA PESTH SUSPENSION BRIDGE.

persons at that time whether piers could be placed in the river by any means. (Fig. 6.)

The drawings reproduced are of coffer-dam No. 3, which was about 72 feet in width and about 136 feet in length inside the puddle walls, there being two puddle chambers, each 5 feet in width. From a point about 13 feet above the clay on which the tower was to rest, was an inside wall of sheet-piling, this space being nearly filled, after excavating, with a bed of concrete. The piling of each row, from 40 to 80 feet in length, was all carefully sized to 15 inches square, shod with iron and driven close together, penetrating 20 feet below the bed of the stream or 40 feet below the zero level. The framing of the ice-breaker and the bracing within the dam was of enormous strength. The number of piles driven in the four coffer-dams reached the enormous total of 5,224, and of the 1,227 driven in dam No. 3, 16¼ per cent. were drawn and redriven. These piles

and the timber were obtained from the forests of Bavaria and Upper Austria. (Fig. 7.)

The first pile on dam No. 3 was set on April 8, 1842, but owing to the difficulties encountered it was not finished until three years later—April 4, 1845. From six to seven days were occupied at the first in driving a pile to a depth of 5 or 6 feet into the clay, but as the work progressed the difficulty increased, the operation of driving one pile consuming from twelve to fourteen days, many piles breaking short off so they could not be withdrawn, and the gravel was dredged out from behind and a second row driven. The report further describes the difficulty of the work: "The dredging for the No. 3 dam was carried on to the average depth of 44 feet from the top of the outer row of piles, leaving about 10 feet of gravel to drive through, and extra piles were driven where the gravel found its way between the piles, as well as where it was known the piles were not driven to the proper depth, or were broken or otherwise injured. As the gravel was dredged out to the above depth, the inner and middle row of piles were driven, and a great part of them got down as was supposed to the requisite depth. The work was carried on in the above manner until the 7th of November, when from the appearance of several piles which were pulled up, and from other causes, it became apparent that the outer row was in a much worse state than had been expected, and it was almost a matter of certainty that those piles which had taken ten or twelve days to get down were not driven to the proper depth by at least 3 or 4 feet, having upset or lost their points to that extent. There was likewise every reason to believe that many of them were broken or dangerously crippled. Added to this the Danube was rising, and at the late time of the year, with winter rapidly approaching, the general appearance of the dam was anything but satisfactory. Upon mature consideration the only course appeared to be to drive a much greater number of piles than was at first calculated upon, and another complete row of piles was driven all around at intervals of 15 inches apart, and in some cases double and triple piles were driven during the progress of the dredging. At the commencement of the driving a few were got down to the depth of 57 or 58 feet, being from 3 to 4 feet in the clay; but as the gravel began to get compressed many of them would not penetrate more than 54 or 55 feet, the sharp, angular gravel overlying the clay appearing to be compressed into a substance as hard as rock."

The puddle used was clay mixed with about one-third clean gravel, it having been found to set quite solid, from experiments made by

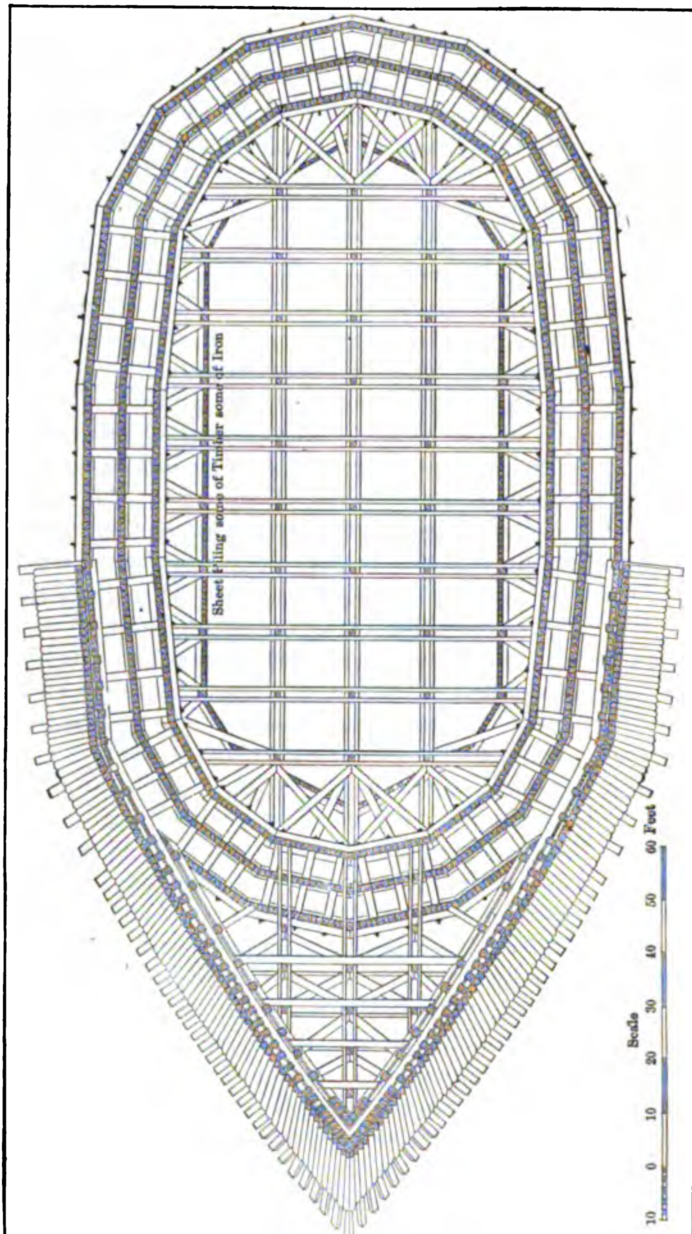


FIG. 7.—BUDA PESTH SUSPENSION BRIDGE, PLAN OF COFFER-DAM NO. 3.

sinking specimens in the Danube. When leaks occurred they were closed by driving square timbers down 30 or 40 feet into the puddle to pack it, or by driving new piles to close the cracks, and in some cases by driving sheet-piling.

Experiences of this nature led to the disuse of coffer-dams for foundations to such depths, but a very small percentage of the care exercised and the persistence shown in this work would lead to greater success on ordinary foundations.

The class of work to which coffer-dams may still be applied will be shown in the succeeding pages and the examples from actual practice will show in some measure the care that must be exercised in the first construction to prevent failure, and the expedients adopted to overcome unavoidable accidents.

The historical features of foundations are of sufficient importance to add to the foregoing pages a résumé of the earliest uses of the various methods described in the subsequent chapters.

Vitruvius is probably the first writer on the subject and he gives instructions as to the methods for making the foundation which can be little improved upon to-day.

"Foundations should be carried down to solid bottom, if such can be found, and they should be built thereon of such thickness as may be necessary for the proper support of that part of the wall standing above the natural level of the ground. They should be of the soundest workmanship, and materials of greater thickness than the walls above. . . .

"The intervals between the foundations brought up under the columns should be either rammed down hard, or arched, so as to prevent the foundation piers from swerving. If solid ground cannot be come to, and the ground be loose or marshy, the place must be excavated, cleared, and either alder, olive, or oak piles, previously charred, must be driven with a machine as close to each other as possible and the intervals between the piles filled with ashes. The heaviest foundations may be laid on such a base."

Thus we find that the use of piles was of a very early date, and rubble mounds for the base of piers, similar to those already mentioned for the bridges of Persia, of very ancient origin. Earthen dams for excluding the water are known to date back to the seventeenth century.

Piles surrounded by rubble stone, in place of the ashes mentioned by Vitruvius, were used at the Notre Dame bridge in Paris between 1500 and 1507. Piles supporting a platform on which to build piers were used at Blois in the year 1716.

Régemortes made use of an apron in preparing a foundation at Moulins in 1750. As early as 1750 caissons were sunk for the piers of the old bridge of Westminster, after the bed of the river had first been dredged, a similar process having been used in 1686, or 64 years earlier, at the Tuileries bridge.

The cutting off of piles under water was accomplished in 1756, by the use of a saw invented by Des Essarts, and this method was used on a great many bridges for the next hundred years.

The discovery in the year 1818 of the properties of hydraulic mortars, by Vicat, made possible the forming of a foundation by depositing concrete inside of sheeting and also the use at Paris by Beaudemoulin for the first time of a bottomless frame or crib, with concrete at the bottom. This method was improved upon by Poirel about 1840, by adding a canvas bottom to the crib, or caisson, to deposit concrete *in situ*.

The earliest account we have of the use of compressed air is given by John Taisner of Hainault, born in 1509, who went to Toledo with the Emperor Charles V, where he saw two Greeks let themselves down under water in an "inverted caldroun" with a light, and return to the surface without getting wet. Lorini also describes the use of the diving bell or the progenitor of the pneumatic caisson, and the first account of its use in England is given by Dr. Halley early in the eighteenth century.

The diving bell used by Smeaton at the Hexham bridge in 1778, is really the first use of compressed air on bridge foundations of which we have an authentic account. His account of it in a letter of instructions gives the details of construction (Fig. 8) and method of using it. The details of the air pump are shown in Fig. 9.

Smeaton employed the diving bell as early as 1778, in the construction of the bridge at Hexham; in a letter which he addressed to Mr. Pickernell on the subject, we have the method he adopted fully explained; he says: "If the cases would have enabled us to reduce the water so low, as to be even with the very bottoms of the caissons of each pier, I take it for granted, you would have thought it no difficulty with broken rubble, beton, stones, and short blocks of wood, cut a little wedgeways, to have crammed and wedged up the cavity washed under the wooden bottoms, so as to have been equally resisting, and capable of bearing a weight with the original gravel, and particularly when this new body of matter is supported, and even jammed tighter into its place by filling up the vacancy between the pier and the base, a little above the wooden bottom, with rubble, and then driving it tight down by a set with the ram.

It therefore now remains that I describe, and make you master of a piece of machinery, that will put you nearly into the same condition, as if the water could have been reduced to the caisson's bottoms as before mentioned; and this is by means of an air-chest, or diving vessel, which being let down, will exclude the water down to the very bottom of the river if you please, and therefore as low as the under side of the wooden bottom, which in the present case is as low as will be necessary or useful, and the chest or vessel being large enough to give liberty for a man to work therein; being furnished with a pair of boots, he will at mid leg deep in water, do his business with almost as much facility as if the water were pumped out to the same level. The principal part of this machine will consist of a strong chest (Fig. 8), suppose 3 feet 6 inches in length, about 4½ feet deep or height and as wide as to give free leave for its going down between the cases and the piers, which I suppose will be about 2 feet wide inside measure, as the other measures are also supposed to be. Now you know very well that if you push a drinking glass, or any other similar vessel, with its mouth downwards into the water, that it will exclude the water, leaving the vessel full of air, as it was before it was

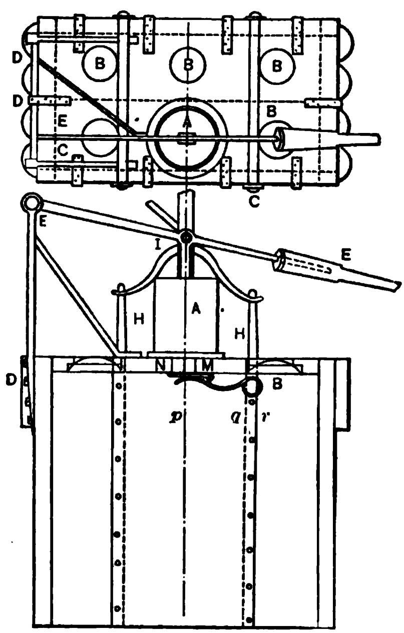


FIG. 8.—SMEATON'S DIVING BELL.

thrust into the water; in like manner, if this chest, being loaded with a sufficient weight, be let down into the water, mouth downwards, the air will exclude the water to the bottom skirt of the chest, and if let down, so as to rest upon the bottom of the river, a man may stand therein, and do any kind of business, the same as he could do in the same space in the open air. But to continue this for any length of time two things are obviously necessary, and those are light and a circulation of fresh air. The former might on occasion be supplied by a candle, but here we may have the advantage of day-light by putting two or three strong round panes

of glass into the bottom of the chest, which will in its inverted situation be the top; a sufficiency of light will enter, this top of the chest being supposed above water. Respecting air, you will conceive that any quantity might be forced in by a strong pair of bellows; but those made of leather would be cumbersome and unhandy; I therefore substitute a kind of foreign air-pump (Fig. 9), made of thin hammered copper, that will throw in a gallon at a stroke, which will not only continually refresh the workmen within, but whatever air escapes through the joints or pores of the air-chest will be replenished, and the overplus go out at the bottom or skirt of the chest, and boil up on the outside.

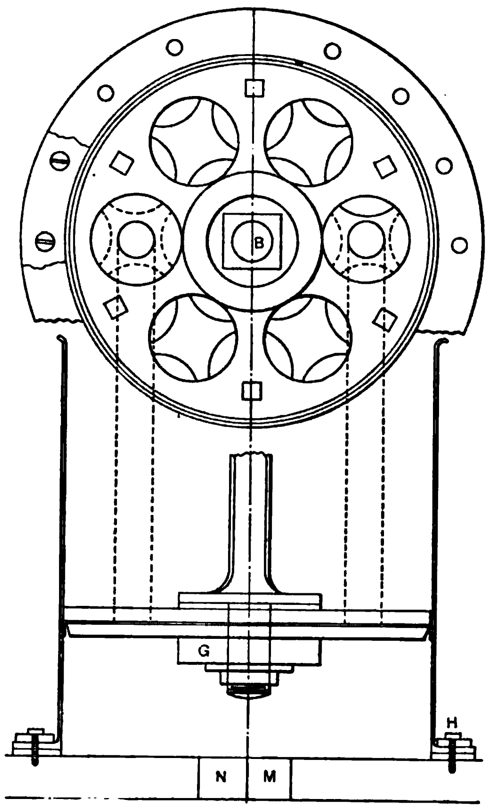


FIG. 9.—SMEATON'S AIR PUMP.

"The quantity of weight that will sink it, mouth downwards, will be the same as placed therein (bottom downward) would sink it the same depth; and as this chest I propose to be suspended by a tackle, and to go down by its own weight, I compute that it will take sixteen pigs of lead to sink it to the bottom of the river, and keep it steady. I propose that the lead may be as much out of the way as possible, to place them upon the ends of the chest, endways upward, that is four in a row below and four above, and the same at the other end, making in the whole 16 pigs, which are to be fastened on with screws, either by cleats screwed on, or punching a hole through each end of each pig. At one end of the chest there is to be a board, fixed across for the man to sit upon, and a cleat nailed to each side to set each of his feet upon, so that while the machine is

being lowered or hoisted, he is totally dry, and when let down enough, he stands upon the bottom of the river, without any more water than the height between the skirt of the chest and the bottom of the river, which may be more or less as is found convenient, I suppose never more than a foot deep, because wherever the ground is taken out more than 1 foot below the underside of the caisson's bottom, I would propose to fill it up with rubble previously to that height or depth, nor can it be of use to let down the skirt of the chest much below the caisson's bottom, because the side of the chest will then diminish the room you will have to get the matter from underpinning the caisson's bottom. The foregoing will, I believe, be sufficient for explaining the general principles and outlines of the method I mean to pursue in underpinning, and resupplying what is underwashed from the bases of the piers, and which I dare say you will now see to be entirely practicable; what you are therefore immediately to put in hand is the air-chest, of or about the inside dimensions before mentioned; I believe the two flat sides will do very well, if of good red wood deal, shot clean of sap, the two ends and bottom, (or in use its top); it would be well if they could be got of single planks of elm, beech, or plane trees, as they would hold the nails better: I fancy $1\frac{1}{2}$ or $1\frac{3}{4}$ inches thick for the sides, $2\frac{1}{4}$ or $2\frac{1}{2}$ for the ends and bottom, will be sufficient; they should be well jointed, and put together with white lead and oil, as the effort will not be of the water to enter, but of the air to escape from within. Were I with you, when it is put in use, I should be the first to go down into it, as there is no more danger (all your tackle being firmly fixed) than being let down into a coal-pit by a rope: and if it shall happen that all your masons are too fine fingered, I fancy a couple of colliers to take turn and turn will find it a very comfortable job. A particular encouragement, must, however, I expect be given. I will give you more particular directions in my next: as to the air pump, all that will be wanted from the coppersmith will be a cylindrical pipe of copper, 10 inches diameter, and 12 inches high, wired at top, and a flanch at bottom of about $1\frac{1}{4}$ inches broad, by which it is screwed down before the top of the air-chest; the copper to be about the thickness of a halfpenny; if you have no neat-handed coppersmith that can hammer it straight and smooth inside, it may on occasion be made of strong tin."

The references to the letters on Figs. 8 and 9 are: *A*, the air pump; *B*, skylights 6 inches in diameter, to be made of window glass knobs, if plate glass cannot be had; *C*, clamp plates of iron, to hold the sides and top firmly together; *D*, *D*, pigs of lead, end

upwards; *E, E,* the lever for working the pump; *G, G,* the axis and brace for steadying the lever; *H, H,* two bows for hoisting the chest; *I,* a strong iron bow to hook the tackle to; *M, N,* the opening from the pump to the air chest; *o, p,* the valve, *o* being the leather and *p* the wood; *q r,* the spring to shut it, having just strength enough to shut the valve.

The following account of the improvement of the diving bell by Sir John Rennie is taken from Smiles' "Lives of the Engineers."

"Whilst occupied on the works of the Ramsgate Harbour of which he was appointed engineer in 1807, Mr. Rennie made use of the diving bell in a manner at once novel and ingenious. It will be remembered that Smeaton had employed this machine in the operations connected with the building of the harbour; but his apparatus being wood, was exceedingly clumsy, and very limited in its uses. In that state Mr. Rennie found it, when he was employed to carry on the extensive repairs of 1813. The east pier-head was gradually giving way and falling into the sea at its most advanced and important point. No time was to be lost in setting about its repair; but from the peculiarly exposed and difficult nature of the situation, this was no easy matter. The depth at the pier-head was from 10 to 16 feet at low water of spring tides; besides, there was a rise of 15 feet at spring and 10 feet at neap tides, with a strong current of from two to three knots an hour setting past it both on the flood and at the ebb. The work was also frequently exposed to a heavy sea, as well as to the risk of vessels striking against it on entering or leaving the harbour.

"Mr. Rennie's first intention was to surround the pier-head by a dam; but the water was too deep and the situation too exposed to admit of this expedient. He then bethought him of employing the diving-bell; but in its then state he found it very little use. No other mode of action, however, presenting itself, he turned his attention to its improvement as the only means of getting down to the work, the necessity for repairing which had become more urgent than ever. Without loss of time he proceeded to design and construct a bell of cast iron, about 6 feet in height, 4½ feet wide, and 6 feet long, having one end rather thicker and heavier than the other, that it might sink lower, and thus enable the exhausted or breathed air more rapidly to escape.

"At the top of the bell, eight solid bull's-eyes of cast glass were fixed, well secured and made water-tight by means of leathern and copper collars covered with white lead, and firmly secured by copper screw bolts. To the top of the inside were attached two

strong chains for the purpose of fastening to them any materials that might be required for the work, and flanges were cast along the sides of the bell, on which two seats were placed, with foot-boards, for the use of the men while working. In the centre of the top was a circular hole, to which a brass-screwed lining was firmly fixed, and into this a brass nozzle was screwed, having a leathern water-tight hose fastened to it, $2\frac{1}{2}$ inches in diameter. The hose was in lengths of about 8 feet, with brass-screwed nozzles at each

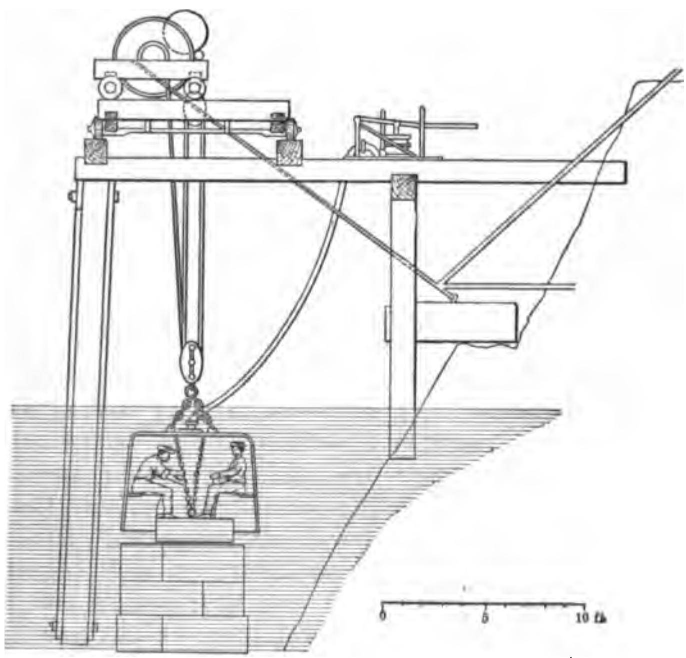


FIG. 10.—RENNIE'S DIVING BELL.

end, so that it could be lengthened or shortened at pleasure, according to the depth of water at which the men in the bell were working.

"For the purpose of duly supplying the machine with air, a double air-pump was provided, which was worked by a sufficient number of men. The air-pump was connected with the hose referred to, and was either placed on the platform above or in a boat which constantly attended the bell while under water. Two stout wrought-iron rings were fixed on the top of the machine to which ropes or chains were attached for the purpose of lowering or raising it. The whole weighed about five tons; and it was attached to a circular framework of timber, strengthened by iron, erected over where the

intended circular pier-head was to be built, and so fixed to a pivot near the centre of the work that it was enabled easily to traverse its outer limits.

"On the top of the framework was a truck, made to move backwards or forwards by means of a rack on the frame, and a corresponding wheel provided with teeth, worked by a handle and pinion. On the truck were placed two powerful double-purchase crabs or windlasses, one for working the diving-bell suspended from it, and the other for lowering stone blocks or other materials required for carrying on the operations at the bottom of the sea. By these ingenious expedients the building apparatus was so contrived as to move all round the new work backwards and forwards, upwards and downwards, so that every part of the wall could be approached and handled by the workmen, no matter at what depth; while the engineer stationed on the pier-head above could at any time ascertain, without descending, whether the builders were proceeding in the right direction, as well as the precise place at which they were at work.

"Everything being in readiness for commencing operations, the divers entered the bell and were cautiously lowered to the place at which the building was to proceed. A code of signals were established by which the workmen could indicate, by striking the side of the bell a certain number of strokes with a hammer, whether they wished it to be moved upward, downward, or horizontally; and also to signal for the descent of materials of any kind. By this means they were enabled, with the assistance of the workmen above, to raise and lower, and place in their proper bed, stones of the heaviest description; and by repeating the process from day to day and from week to week the work was accomplished with as much exactness, and almost as much expedition, under water, as though it had been carried on above ground.

"Thus the entire repairs were completed by the 9th of July, 1814; and to commemorate the ingenuity and skill with which Mr. Rennie had overcome the extraordinary difficulties of the undertaking, the trustees of the harbour caused a memorial stone to be fixed in the centre of the new pier-head, bearing a bronze plate, on which were briefly recorded the facts above referred to, and acknowledging the obligation of the trustees to their engineer. They also presented him at a public entertainment with a handsome piece of plate in commemoration of the successful completion of the work. The diving-bell, as thus improved by Mr. Rennie, has since been extensively employed in similar works; and although detached divers,

with apparatus attached to them, are made use of in deep-sea works, the simplicity, economy, and expeditiousness of the plan invented by Mr. Rennie, and afterwards improved by himself, continue to recommend it for adoption in all undertakings of a similar character."

The first use of compressed-air caissons was by M. Triger from 1839 to 1841 at the Chalons coal mine, and this was rapidly improved upon for foundation work, until we have the modern compressed-air caisson.

Dr. Potts brought out his vacuum process in 1845, in which the air is exhausted from the caisson, and the external air pressure utilized for the weight to sink it; this process, however, is seldom if ever used at the present time.

One of the earliest cases of the use of the pneumatic process for sinking bridge piers in America was at Omaha over the Missouri River in the year 1869, under the direction of Gen. Wm: Sooy Smith. The piers of the St. Louis Eads bridge and of the East River Roebling bridge were sunk by this process between 1870 and 1873.

"In every man's mind, some images, words, and facts remain, without effort on his part to imprint them, which others forget, and afterwards these illustrate to him important laws."

CHAPTER II

CONSTRUCTION AND PRACTICE—CRIB COFFER-DAMS

THE exact definition of the term coffer-dam—" a water-tight inclosure, from which the water is pumped to expose the bottom and permit the laying of foundations "—is the class of structure which is to be considered, although in the construction of them cribs or caissons may be employed and utilized; the essential purpose being to form an inclosure as nearly water-tight as possible in order that the expenditure of power for pumping out the water may be of small amount.

The attainment of this when the water is shallow and has little current we have seen to be easily accomplished by means of a bank of clay or clayey gravel.

This form may also be employed in still water up to about 4 feet in depth by the addition of sheet-piling or a casing supported by ordinary piles to prevent the embankment from caving into the excavation. Where the bottom is of soft mud or porous material over a solid clay or gravel, as much as possible of the porous material should be removed before forming the embankment, thus preventing leakage underneath. In very shallow water this can be accomplished by shoveling and with large hoes or scoops, but with several feet of water to contend with, some form of dredge or scraper must be employed. A very convenient form of scraper used by M. L. Byers on the Cinti. & Mus. Valley Railway is described in Vol. 31 of the " Transactions of the American Society of Civil Engineers," and consists of old boiler-iron, strengthened by three ribs of light iron rail as shown in Fig. 11. This was operated by a double-drum 20-horse-power Mundy hoisting-engine, with the towing-line running directly from one drum to the scraper and the back line from the other drum over a sheave to the front of the scraper. The excavating averaged about 45 yards of material each day during twelve days' work. The weight of the device was about one thousand pounds.

Where the material is very soft, a hand-dredge, called a spoon, will accomplish the work at about the same cost as excavating on dry land. The spoon usually consists of a long pole, having a cutting-ring fastened at one end, and to this ring is attached a canvas bag to contain the excavated material. The ring is hung from a derrick with a set of falls, being guided with the pole, as it is dragged forward by the derrick through the material to be excavated.

Excavating may be done on all the larger rivers by employing the sand or gravel diggers which are almost always to be found, the dredging being accomplished by means of a series of buckets on a

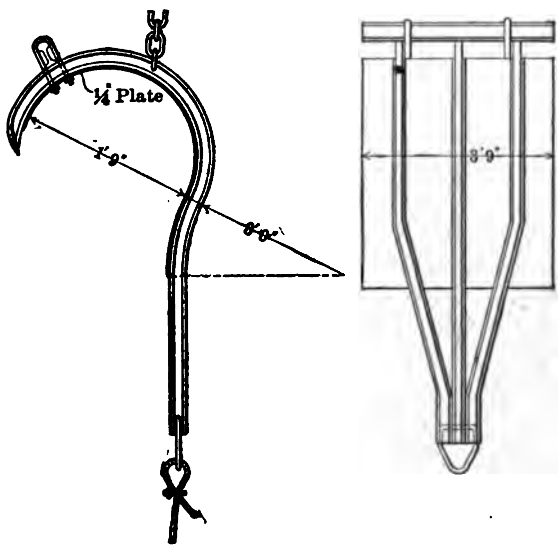


FIG. 11.—SCRAPER DREDGE.

belt or on chains operated through a well in the bottom of a barge. Dredging by machinery on a large scale will be considered later on in some detail.

The method of embankment is sometimes employed for greater depths than 4 feet and in some instances successfully.

The Chanoine dams on the Great Kanawha River required substantial foundations beneath the water, and to accomplish this Addison M. Scott, the resident engineer, employed log cribs about the spaces, with earth banked up on the outside. This work is described in the report of the Chief of Engineers for 1896, the metal work for dams Nos. 9, 10 and 11 being constructed under direction of the author as Chief Engineer of the fabricating company.

The site of the navigation pass of dam No. 11, including the center pier, required a coffer-dam 90 feet wide and 330 feet long inside. (Fig. 12.) This area, including the necessary room for the cribs, was dredged out to hard-pan from 20 to 24 feet below low water. The log cribs, which contained about 84,000 lineal feet of logs, were sunk in sections 19 feet wide and 20 feet long. They were sheathed up to about 3 feet above low water, with sheet-piling in three layers, on the Wakefield system. The driving was accom-

FIG. 12.—COFFER-DAM AT DAM NO. 11, GREAT KANAWHA RIVER.

plished by attaching an eighty-pound weight to an Ingersoll-Sergeant drill run by steam and utilizing the reciprocating motion by attaching the drill with clamps to the tops of the sheathing, following it down as it was driven, after the manner of the Nasmyth steam pile-hammer. This tool, which is one of the most ingenious ever devised for the purpose, was arranged by the contractor's engineer, S. H. Reynolds, and was a complete success.

The tops of the cribs were 10 feet above low water, and the bottoms rested on the hardpan, making a total height of from 30 to 34 feet.

The cribs were filled with sand and gravel that had been dredged out, but the outside was banked up with selected clay and dredged material, which was protected by a layer of riprap up to about low water.

When the coffer-dam was first pumped out several leaks were developed, but after one week in perfecting the details the pumps were started regularly and no serious trouble was had afterward. This is only one of a series of coffer-dams which have been constructed on the several dams in this river, and owing to the care exercised good results were obtained uniformly.

The construction of a similar piece of work on the Ohio River was begun by Major R. L. Hoxie, corps of engineers, and is described in the report of 1895: "It was originally planned to inclose the site of the dam and lock within a coffer-dam, and work was commenced upon that basis. But on attempting to pump out the inclosure it was found that water came in in large quantities, not only under the dam but from springs in the bottom, and all attempts to close these by dumping clay and gravel were a failure. The area inclosed by the dam was about 600 by 200 feet or about 3 acres of river bottom. The deposit of sand and gravel overlying the rock was about 35 feet thick, the rock being 45 feet below the water-level, while the plans required an excavation 20 feet deep below this water-surface. The bottom deposit had been worked over for years by sand-diggers, who threw back the large stones and coarse gravel after removing the fine sand, this work resulting in a very permeable bottom, with possible channels of comparatively large dimensions extending to unknown distances beyond the limits of the coffer-dams."

This is perhaps the most frequent source of failure of a well-constructed coffer-dam and should always be guarded against by removing as much of the porous material as possible, by dredging, before the construction of the coffer-dam is begun.

Cribs are very easy to construct, usually very substantial, and easy to make use of by floating to position and then sinking in place. A very simple form that has been used on the Chicago, Burlington & Quincy Railroad is described by E. J. Blake, chief engineer. Where the water is shallow they have been built in the form shown (Fig. 13), of fence boards spiked one piece on another; with deeper water they are made of heavier timber, $2'' \times 8''$ or $2'' \times 10''$. They are built on the water and are tied across at intervals by pieces spiked through the wall, which pieces should be carefully fitted to prevent leakage. In some cases where the bottom is soft, instead of dredg-

ing, a bottom is added to the crib to prevent the filling from squeezing its way out from under the edge.

When the crib has reached bottom, being sunk by weighting it down if necessary, the chambers are filled with clay puddle and clay is banked up around the outside to prevent water running under. The crib is made large enough so that the excavation will leave an easy slope to the inner edge of the timber work. This form can be made to conform readily to the contour of the bottom by starting the layers of timber at different elevations. No leakage

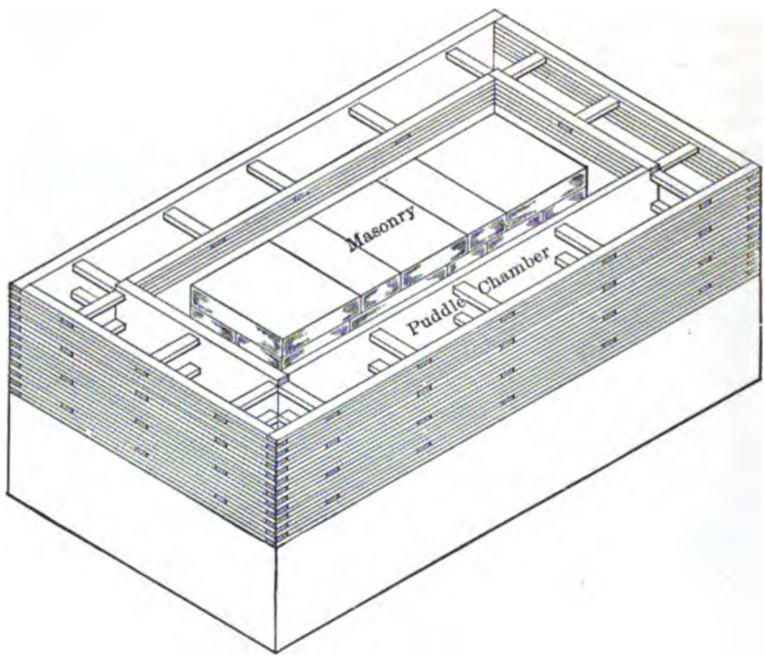


FIG. 13.—CRIB COFFER-DAM, CHICAGO, BURLINGTON AND QUINCY RAILROAD.

has been experienced except what can readily be kept under control with ordinary-sized centrifugal pumps. The cost of construction is generally a minimum, as there are usually plenty of old timbers available for use from the railroad yards.

Cribs constructed in a similar manner but with only one wall of timber have been used successfully on the Canadian Pacific Railway by P. Alex. Peterson, chief engineer.

The bracing is very efficiently attached by dovetailing it into the sides, while the form of the crib enables it to withstand the force of the current and the ice. The projections on the inside are to

prevent the water from forcing its way up between the sides and the concrete filling when the dam is pumped out. These projections answered their purpose very effectually, and when the dam was

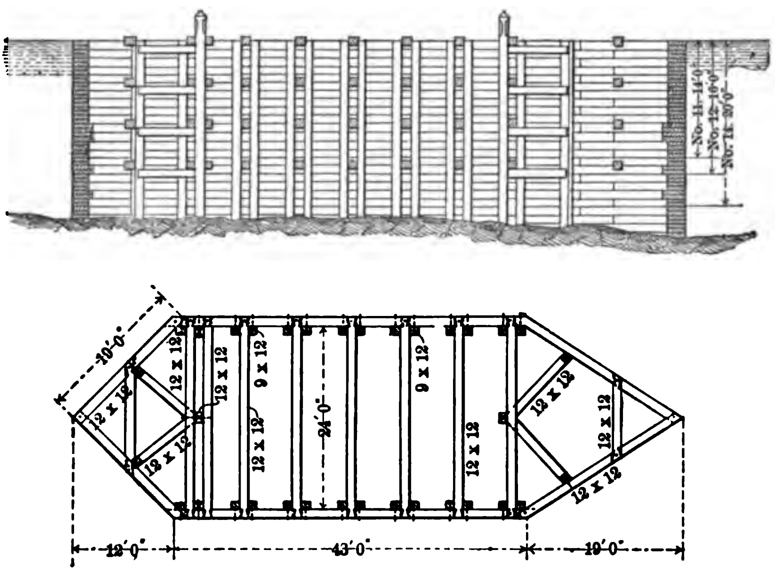


FIG. 14.—ST. LAWRENCE RIVER BRIDGE, CRIB AND COFFER-DAM, CANADIAN PACIFIC RAILWAY.

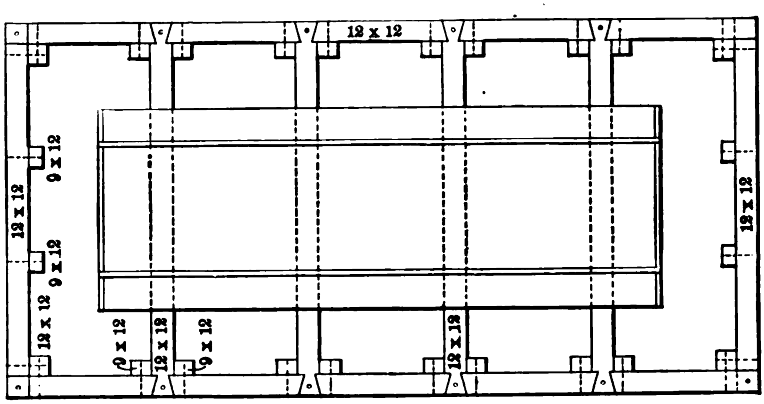


FIG. 15.—ARNPRIOR BRIDGE, CRIB AND COFFER-DAM, CANADIAN PACIFIC RAILWAY.

pumped out it remained dry enough to lay the masonry without any additional pumping.

Illustrations are given of a crib of this character which was used on the St. Lawernce River (Fig. 14), similar ones being used for the

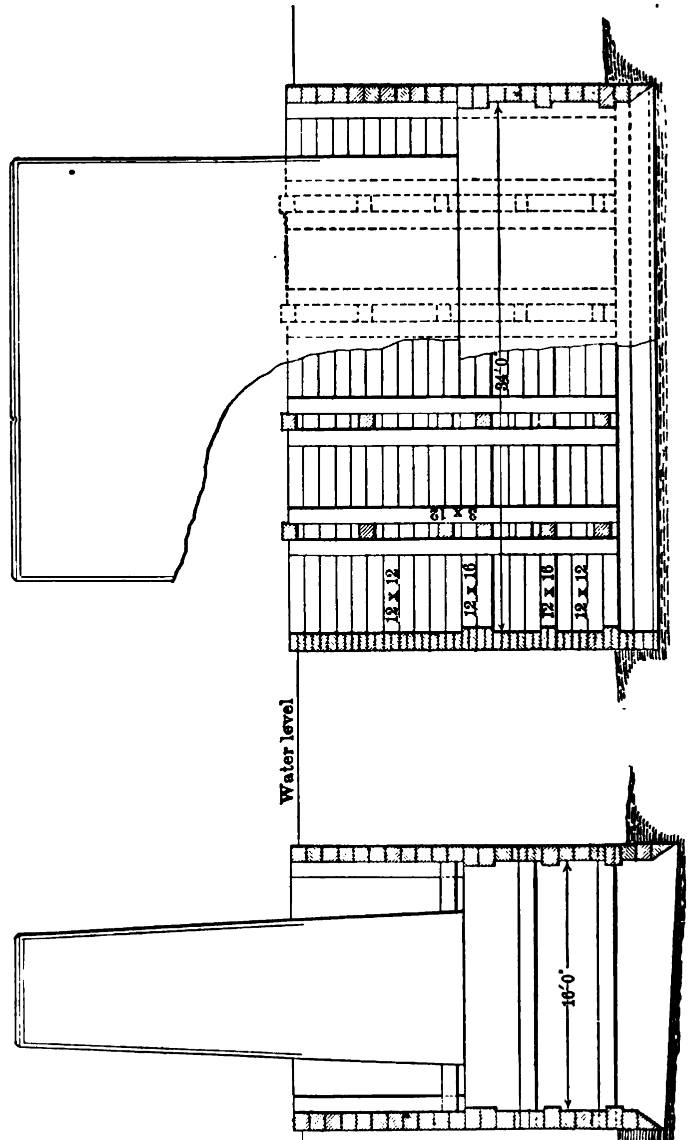


FIG. 16.—ARNPRIOR BRIDGE, CRIB AND COFFER-DAM.

other piers of the same bridge, and of the crib used for the Arnprior bridge. (Figs. 15, 16.) This shows the concrete which was deposited on which to found the masonry, and which formed a water-tight bottom so that the crib could be pumped out for the laying of the stone.

The practice on the Atchison, Topeka & Santa Fé Railroad has been in some respects similar to what has been given. C. D. Purdon, assistant chief engineer, states that cribs built of old timbers are used when such material as stringers $7'' \times 16''$ is plentiful, each course being stepped in about $\frac{1}{2}$ inch to give a batter. For use in sand when rocks and drift are likely to be encountered a crib is made by constructing a frame of old bridge timbers and sheathing it with plank. (Fig. 17.) This is sunk by digging out the sand, which is shoveled first into box *A*, then to boxes *B*, then to *C*, and then outside. The suction-pipe is shown in dotted lines, the pumping being accomplished with a centrifugal pump. This plan works very successfully on the streams in Colorado and New Mexico where the water is mostly in the sand and but little shows as surface-water.

The Arkansas River bridge of the St. Louis & San Francisco Railroad at Tulsa was built over a bottom of gravel and riprap above rock, which was quite level and about 7 feet below low water. Cribs were constructed for coffer-dams similar to the one just described and set on the bed of the stream. Clay from the bank was dumped outside and as the crib was dug out and sunk, the clay followed down and kept out the water.

When the bottom is of clay or of sand without obstructions, sheet-piles, either tongue and groove or the Wakefield, are driven around a crib.

Geo. H. Pegram, chief engineer of the Union Pacific system, has made the construction of coffer-dams conform to available material and local conditions. At the crossing of the Republican River in Kansas, where the bottom was sandy, a single thickness of 4-inch V-shaped tongue-and-groove sheet-piling, with the usual guide-piles and wales, served to form a water-tight structure.

Where a gravel bottom overlaid a hard soapstone, as on some work in Idaho, with 7 feet of water to contend with, the coffer-dam was made of Wakefield piling, formed of $1\frac{1}{2}$-inch sized plank. The joints were tightened with cement; and sand, gravel, and straw placed outside to prevent leaking. Wakefield piling has also been used for clean rock bottom, placed in two rows about the depth of the water apart. Intermediate cribs filled with rock were used to sink them. The ends of the piling were sharpened and driven on the rock until broomed up and rendered nearly water-tight, when

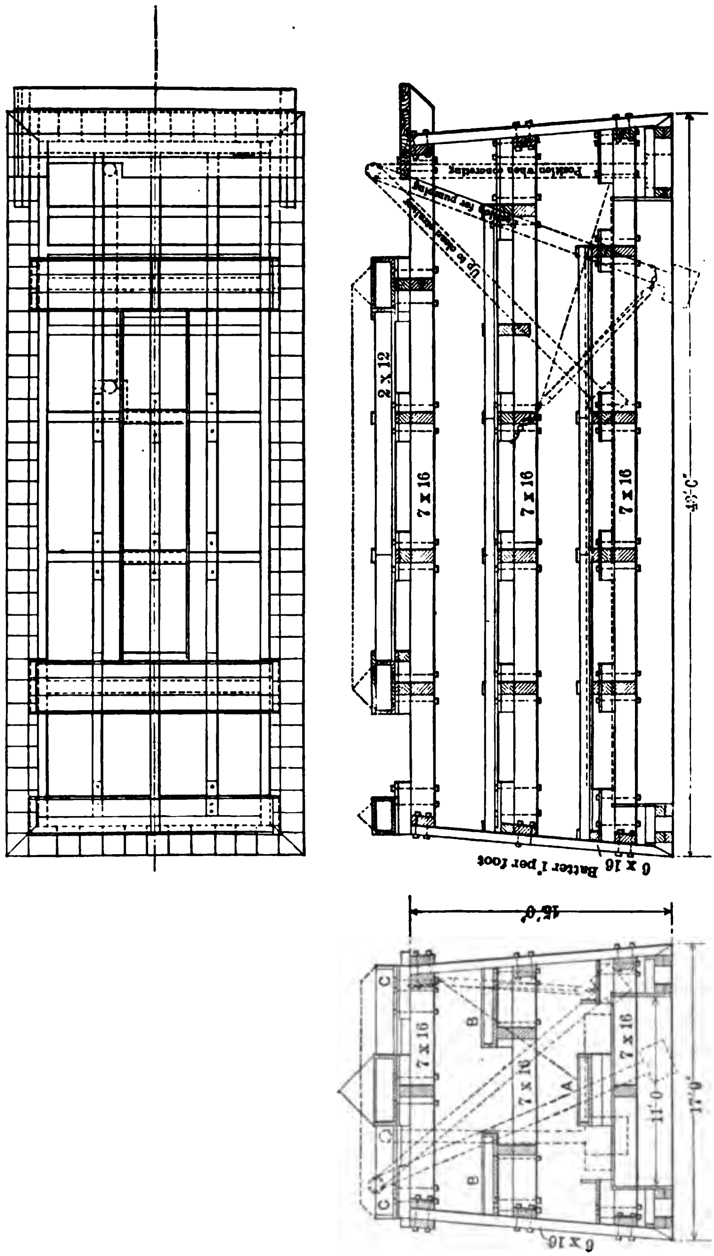


FIG. 17.—CRIB COFFER-DAM, ATCHISON, TOPEKA AND SANTA FE RAILWAY.

gravel mixed with straw was placed around outside to close any remaining leaks.

In cases where ordinary piling has been driven and a grillage laid upon them to receive the masonry, a coffer-dam is constructed as shown (Fig. 18) in which to lay the masonry. The construction of this is fully shown in the different views given.

Another form of coffer-dam for the same purpose was constructed by Octave Chanute in laying the masonry of the pivot pier for the Fort Madison bridge over the Mississippi River, on the line of the Atchison, Topeka & Santa Fé Railroad. (Fig. 19.) This is described in the *Engineering News* of June 2, 1888, by W. W. Curtis, resident engineer: "The grillage (for the pivot pier) is 4 feet 3 inches thick, the upper 15 inches being dressed to an accurate circle of the desired diameter. The coffer-dam was fitted against these two courses and was formed of $3'' \times 8''$ pine-plank staves, dressed on the sides to a slight bevel, around which were placed seven wrought-iron hoops $4'' \times \frac{3}{16}''$, $5'' \times \frac{3}{16}''$, and $6'' \times \frac{3}{16}''$, similar to those used for water-tanks, and screwed up tight. Inside of these, circular braces of plank were fitted. As a water pressure of 19 feet was to be resisted, additional security against leakage was obtained by placing a string of candle-wicking vertically between each stave. When the caisson was submerged to about full depth it became necessary for the steamboat to assist it into final position. A $12'' \times 12''$ post was bedded in the concrete in the center of the pier, with four braces running to the circular bracing of the sides. This makes a very cheap coffer-dam and was found to work very well."

An attempt to use a form similar to this was made in constructing the Walnut street bridge at Philadelphia. This is described by Geo. S. Webster, chief engineer Bureau of Surveys, in the *Engineering News* of March 15, 1894: "In founding the river piers, the Robinson coffer-dam was first tried, but was abandoned after three of them had failed by collapsing. This dam may be briefly described as follows: A circular platform about 80 feet in diameter supported upon piles at an elevation of about 4 feet above high water was first constructed. Square piles of $12'' \times 12''$ yellow pine were then prepared by spiking a $3'' \times 4''$ timber flat, along the middle of one side, and two others along the edges of the opposite side, forming a tongue and groove on each pile. The tops were squared off and the bottom ends pointed to a wedge shape. These piles were then driven close together against the edge of the circular platform and down to rock. Mr. Robinson's idea was that the mud overlying the rock would hold the piles in position at the bottom,

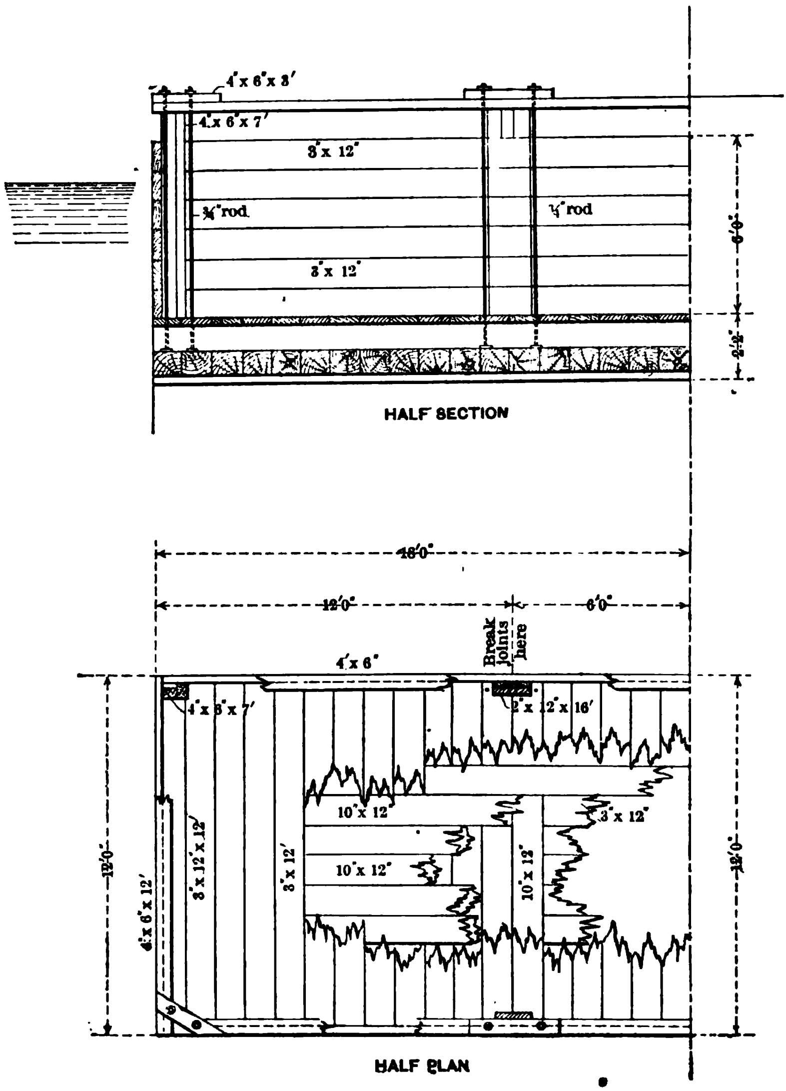
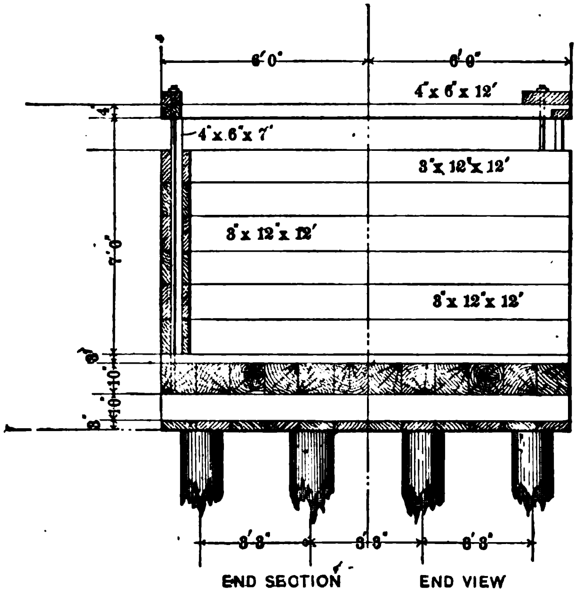


FIG. 18.—COFFER-DAM ON GRILLAGE, PAYETTE AND WEISER RIVER BRIDGES, UNION PACIFIC SYSTEM.



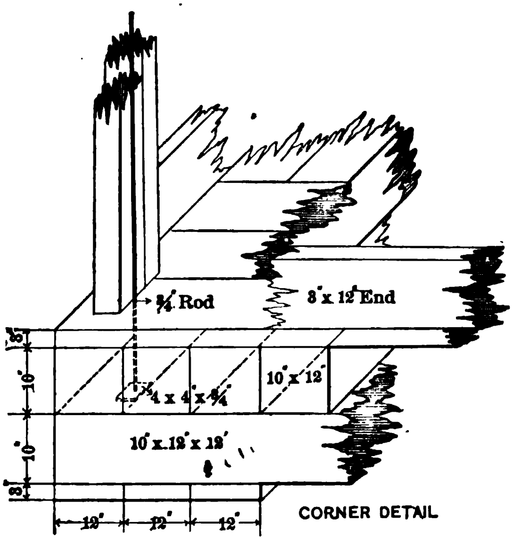


FIG. 18.—CONTINUED.

and if the top ends were held by an outside hoop, the dam would be secure without internal bracing to resist collapsing-pressure. In the first trial the hoop was made of boiler-iron some 4 feet or more in width. In the second dam it was formed of a heavy steel railway rail, and in the third dam the hoop was the same as in the second, but it also had a number of radial rods in addition. The first dam was pumped out and held for nearly an hour before collapsing, but the others collapsed before being entirely pumped out. After the third failure this form of dam was abandoned."

It would seem likely from a comparison of the two cases, one being entirely successful and the other a failure, that had the Wal-

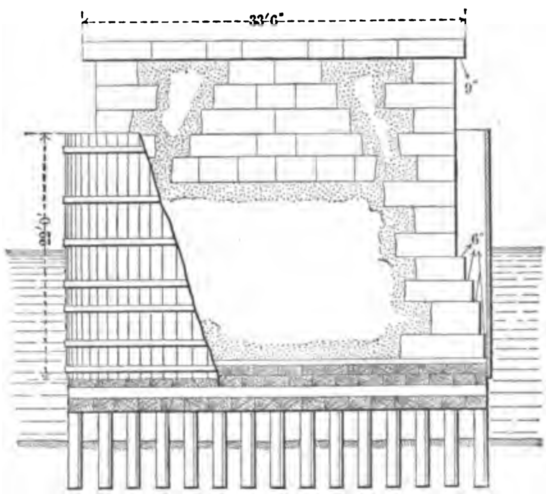


FIG. 19.—COFFER-DAM ON GRILLAGE, FORT MADISON BRIDGE, ATCHISON, TOPEKA AND SANTA FE RAILWAY.

nut street dam been supplied with additional bands lower down and provided with some means of tightening, with several internal bracing ribs of timber, it would have proven a success. These bands and ribs could likely have been placed by a diver.

The uncertainty which always exists regarding any construction under water makes it imperative that every precaution should be taken to guard against troubles that might arise, by making the construction of no doubtful form and in no doubtful manner from its first inception.

The nature of the bottom will always indicate the method of construction which should be adopted in a given case, but it would

be rarely that the preliminary dredging could be dispensed with. It is true that there are cases where there is a deposit overlaying a seamy rock, and the water will find its way along the seams, bubbling up in springs inside. Recourse must be had to cutting off the flow, by puddling on the outside, sometimes extending the operations a distance of a hundred feet or more away, until enough of the flow has been stopped so that the water can be kept down by a reasonable amount of pumping.

The next precaution after dredging is the building of some form of coffer-dam which shall effectually exclude any flow through the

FIG. 20.—A CRIB COFFER-DAM AFTER A FLOOD.

sides of the dam. This we have seen to be accomplished in many cases by means of a bank of clay, or a row of sheet-piling, and in some cases by a single-walled crib. But in the last two methods a supplementary bank of clay or clayey gravel on the outside is necessary to prevent leakage.

This bank may be protected from wash by covering it with clay, sand, or gravel in gunny-sacks, or by riprapping up to about low water, as was done on the Kanawha dams.

Double-walled cribs and coffer-dams, constructed with two rows of water-tight sheet-piling, require to be puddled with a carefully selected material. While clay can be used with a good degree of success, it will be found better to use a clayey gravel or to mix the

clay and gravel, as was done at the Buda Pesth bridge. When a small leak starts through a pure clay puddle, it washes out the clay in considerable quantities and a dangerous leak is soon developed. With the admixture of gravel, however, a leak is stopped almost as quickly as started by the heavier gravel falling into and closing the void.

It will generally be found advantageous to use a bank of clay outside of a double-walled dam, unless it might be a case where sheet-piling has been driven to rock, and even then a certain amount of material in sacks should be used to prevent wash or the cutting out of the earth around the sheeting.

Whatever excavation is taken out of the interior of the coffer-dam after it has been pumped, should be dumped at the up-stream end and corners, or to fill any holes or pockets there may be around the sides or ends.

Cutwaters should be added to all coffer-dams which are built in rivers having a swift current or a heavy flow of ice, as was the case at Buda Pesth and on the Canadian Pacific examples. They must also be used in rivers where the run of drift with each rise is of large amount. For the purpose of preventing wash around a dam, a cutwater of plank supported by a frame of timber may be constructed separate from the main structure, or a V-shaped row of sheet-piling driven up-stream. On rock, a timber crib of triangular shape, built of round logs, may be sunk up-stream and filled with broken stone. Such a crib can be utilized in anchoring the main crib of a coffer-dam, as was done at St. Louis, and which will be described in future pages.

The use of a log crib by the author, somewhat similar to those used on the Great Kanawha River, was employed in placing a reinforced concrete pipe 112 inches in exterior diameter under the bed of Green River for the Tacoma, Washington, water system.

The location (Fig. 21) was at a narrow point in the river where there was considerable fall in a short distance, and the first cribs enclosed about one-third of the width of the river.

The logs were halved into each other and drift-bolted together with $\frac{3}{4}$-inch drift-bolts; the filling was composed of the muck from a tunnel on one side of the river immediately adjoining, comprising broken rock, rock dust, gravel and some red clay. The coffer-dam having been set directly on the bed of the river without any previous excavation, there was some seepage underneath the dam at the junction with the river bed. To take care of this an inside sack dam was built, to carry this water around to the low side of

the coffer-dam, where the fall of the river in that distance made it possible to discharge it into the stream.

The coffer-dam had a height of from 4 feet near the shore to 6 or 8 feet out in the stream, and for probably two-thirds of the depth excavated, a 6-inch centrifugal pump kept the water out, but as the depth increased up to 15 to 18 feet the water came in through the pervious gravel bed of the river to such an extent that an additional 6-inch pump, operated by a 16 horse-power gasoline engine, was required to keep it dry enough to work in.

FIG. 21.—GREEN RIVER LOG CRIB COFFER-DAM.

On the river side some sand bags and some Wakefield sheet-piling were used to prevent caving in of the excavation.

The placing of the pipe (Fig. 22) was accomplished by setting the circular inside forms on concrete blocks, which were buried in the concrete of the pipe when it was poured. Then the reinforcing was placed and wired together and the outside forms set ready for the concrete, which was deposited from the mixer directly on the work by wheelbarrows, the water being excluded except a small portion at the bottom and which was forced out ahead of the concrete as it was poured.

The balance of the river on the other side was included in the

other portion of the coffer-dam, after the pipe had been back-filled and riprapped, this allowing the river to be turned through. The cost of this work will be given in the chapters on cost at the end of this volume.

More fitting language cannot be found for closing words than those used in Wellington's monumental work on railway location: "The uncertainty as to the exact requirements to be fulfilled by the works when completed is a disadvantage, indeed, which cannot be escaped;

FIG. 22.—PLACING REINFORCED PIPE, GREEN RIVER COFFER-DAM.

but the more difficult it is to reach absolute correctness, the greater need we have of some guide which shall reduce the unavoidable guess-work to its lowest terms, and so save us from the manifold hazards which result from not only guessing at facts, but at the effect of those facts. Whatever care we use we can never attempt with success to fix the exact point where economy ends and extravagance begins; but what we can do is to establish certain narrow limits in either direction, somewhere within which lies the truth, and anywhere outside of which lies a certainty of error."

CHAPTER III

CONSTRUCTION AND PRACTICE—CRIBS AND CANVAS

When for some reason the necessary care was not exercised in the construction of a coffer-dam and in puddling it, or where there were discovered conditions not known before the construction began, which rendered the work unsatisfactory or leaky, it will usually be found that the mode of repair which seems most expensive will in the end prove the cheapest and most expeditious. If the puddle proves leaky, and it be decided that the material was of too porous a nature, the best remedy is to dig out and replace it with better. Should it be found that the porous bottom had not been removed to a sufficient depth, it may be found necessary to dig out the puddle-chambers and puddle deeper, or the leaks might be stopped by banking up outside of the dam with clay or clayey gravel, or perhaps sand in sacks would do some good.

Gravel will allow the percolation of water even where the head is small, and when a pressure of from 4 feet upwards is brought upon it, the leakage becomes considerable and difficult to control, so that pure gravel is of little service in stopping leaks.

Hay, straw, oats, crushed cane-stalks, rotten stable manure, and similar materials, mixed with the banking material, are very efficacious in producing tightness, and when applied to local leaks will assist in closing them.

Where sheet-piling has been used to exclude the water and leaks still occur, they can often be closed by driving more sheeting to lap the cracks, which may have been widened out lower down as the sheet-piles were first driven. This, we have seen, produced satisfactory results at Buda Pesth, where leaks were also closed by driving square timbers into the puddle to compact it.

Clay can also be forced down through pipes directly to where the leakage occurs. The use of this at the Government Lock at Sault Ste. Marie is described in the *Engineering News* of September 26, 1896: "The only difficulty encountered in the work of excavation was due to a leak in the coffer-dam, which flooded the lock-pit and

delayed the work considerably. The cause of this leak was found to be a crevice in the rock passing underneath the coffer-dam, and despite all efforts to close it, the flow of water rapidly enlarged the break until about 50 feet of the clay in the coffer-dam had been washed away. The large break was closed by driving additional sheet-piling and filling in with brush, hay, and clay in sacks. This, however, failed to entirely stop the leak through the crevice, and it was determined to fill the cavity with clay. For this purpose a 3-inch pipe was driven down through the coffer-dam until its lower end penetrated the crevice. In this pipe small cylinders of clay about one foot long were placed and forced down into the cavity by means of a plunger working in the pipe. The apparatus is shown in the illustration (Fig. 23). As will be seen, the plunger, or rammer, is an iron rod, to the top of which is fastened a block of wood sliding between the guides of an ordinary pile-driver. The hammer of the pile-driver is the weight which pushes down the rammer. This apparatus was designed by E. S. Wheeler, engineer in charge of the work, and was used not only to fill the crevice, but all along the coffer-dam for the purpose of compacting the clay filling. The apparatus proved most successful for the purpose for which it was intended."

The use of rods for bracing in double-walled coffer-dams is very often the cause of considerable leakage, the water following along them through the puddle. This may be stopped by wrapping a band of hay or straw around the rod next to the timbers, or by a wrapping of coarse cloth, or by a wood washer having a hole slightly smaller than the rod, which is forced through.

The walls of the dam must always be made tight, and this we have seen to be effected by careful framing of sides and bracing, and it will be seen in a later example how round struts between the two walls allowed the puddle to flow around them and close up much better than if the braces were square timbers.

The use of candle-wicking between the staves proved successful at Fort Madison, and calking is very often resorted to at the first, and also to close up local leaks. The use of this and the use of a stiff grease between the layers of a crib will be referred to in another part of this article.

The use of tarpaulins to make a water-tight piece of work is described in the Trans. Am. Soc. C. E., Vol. 31, by Montgomery Meigs, engineer in charge of the government work at Keokuk, Iowa. "The upper one of three locks was twice repaired by separating it from the river by an ordinary plank and mud coffer-dam. But as this

work had to be done after the close of navigation, it was found to be very unsatisfactory on account of the freezing of the puddle, and on one occasion the partly puddled dam froze and upset. After this experience it was determined to use some other method than

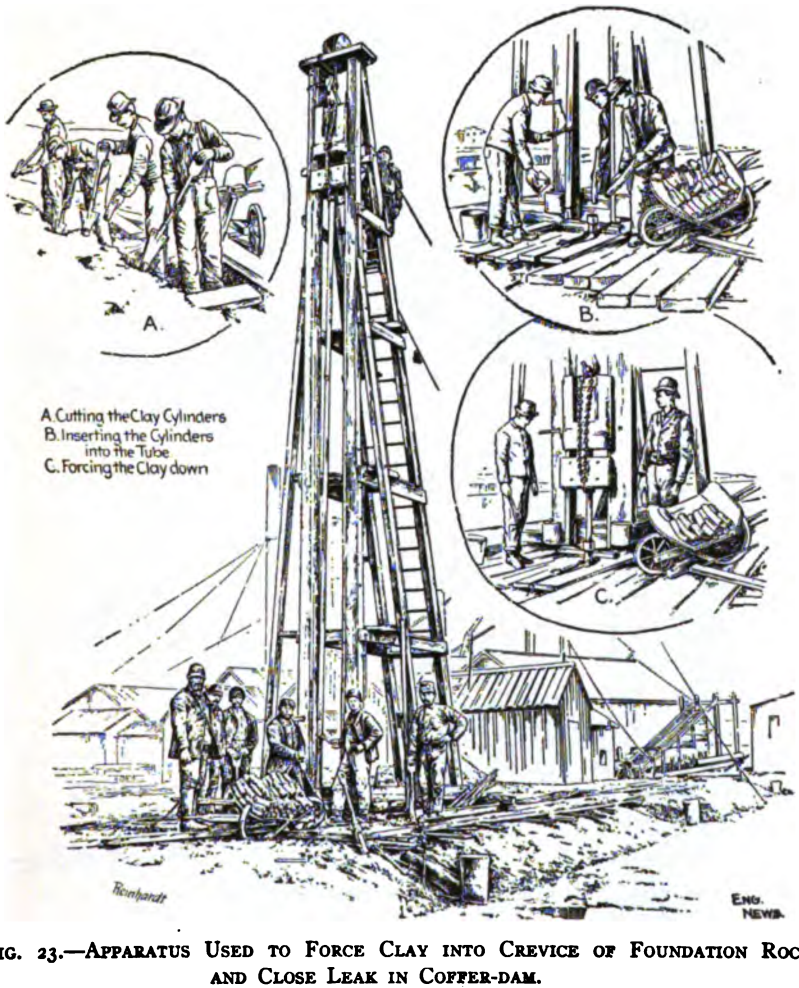


FIG. 23.—APPARATUS USED TO FORCE CLAY INTO CREVICE OF FOUNDATION ROCK AND CLOSE LEAK IN COFFER-DAM.

puddle to produce tightness. There was available for drainage a 50-H.P. suction dredge, with 14-inch suction, and a rotary Van Wie pump, and plenty of 12-inch discharge-pipe mounted on pontoons. It was proposed to drain the lock with this dredge, allowing the boat to settle in the mud at the bottom of the lock as the water

left it, and to complete the work with a 3-inch discharge pulsometer. The lock being 350 feet long and 80 feet wide, a flat place on the bottom was selected, the dredge placed over it and the necessary length of discharge-pipe placed in position on its pontoons. The point selected for a bulk-head (Figs. 24 and 25) was just outside the lock gates, about 40 feet below the lower miter-sill, where there

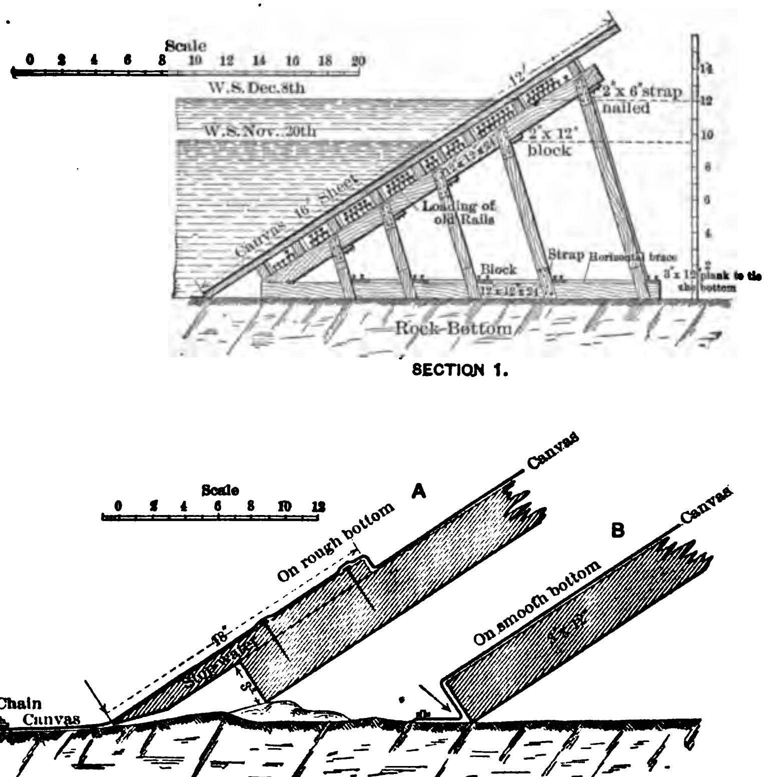


FIG. 24.—DETAILS OF CANVAS AND PLANK BULKHEAD.

was a smooth rock bottom, the ends of the dam abutting against the flaring ashlar wing-walls of the lock approach.

"The bulkhead was constructed with thirteen bents 8 feet apart, of the size timber shown, with light diagonal bracing. After being built 2½ miles from the lock it was towed to position and sunk by weighting it with old railroad-rails, enough being used to overcome the buoyancy after the sheathing was added. A diver was employed to see that the bottom was clear of obstructions and to guide the

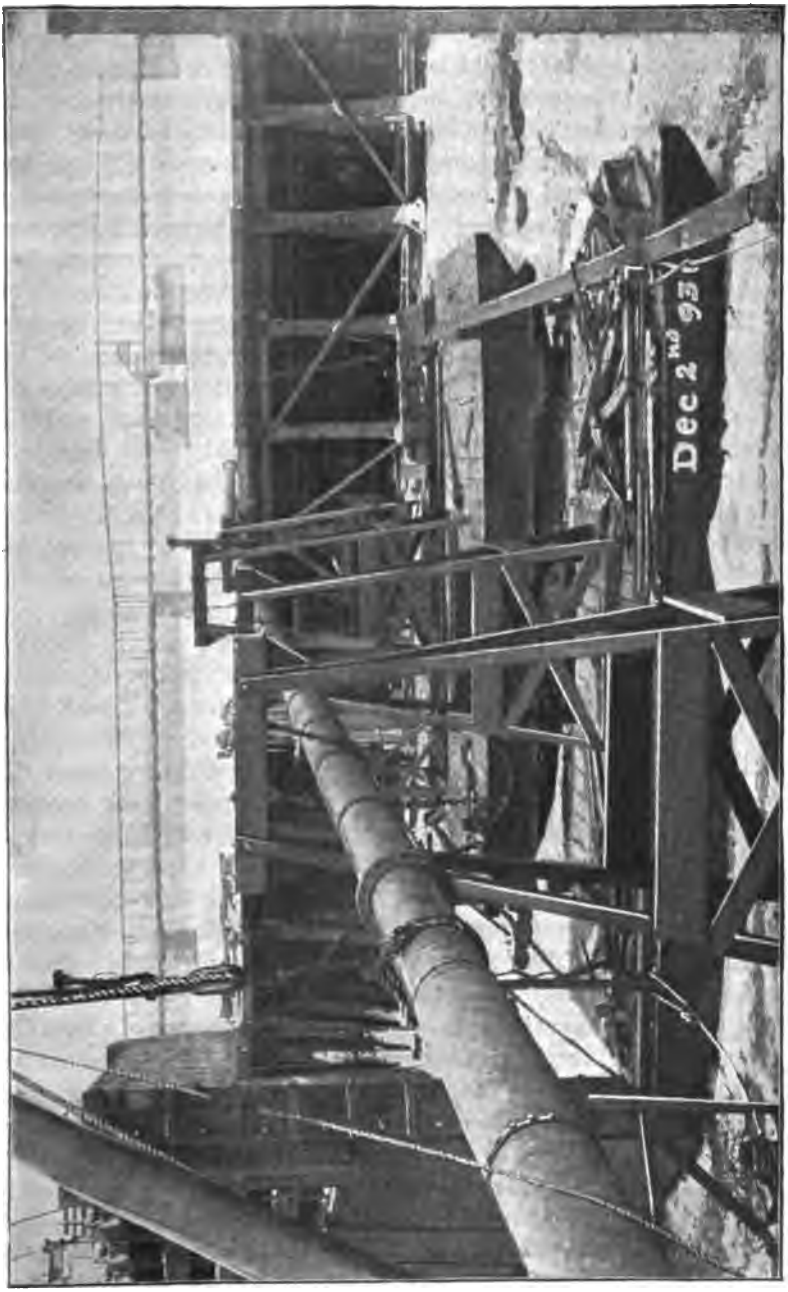


FIG. 25.—INSIDE VIEW OF BULKHEAD. LOCK PUMPED DRY.

bulkhead to a solid bearing. The sheathing was also guided to place by his assistance.

"The canvas sheet, which was designed to give tightness to the apron, was of two breadths of 10 feet and one breadth of 6 feet wide, sewed together edge to edge for convenience, and about 4 feet longer than the extreme length of the apron. Some old $\frac{1}{2}$-inch and $\frac{5}{8}$-inch chain was sewed to one edge continuously to act as a sinker and insure the lower edge of the canvas sheet hugging the bottom tightly. A few stones laid on it would have answered the same purpose, but not so well. The canvas was 12-ounce duck.

"The sheet was spread under water by the diver. It lapped on the bottom about 12 inches, covered the face of the apron and extended some inches up the face of the wing-walls at the end of the dam. Cleats were nailed on the angle between the apron and the wing-walls. These were of 1×4-inch strips, nailed with 2-inch wire nails about 12 inches apart. The upper edge of the canvas was also lightly cleated to the planking in a similar manner. No other nails were driven in the canvas, which was designed to be cut up into tarpaulins eventually. Where the plank touched bottom no beveling was used, but one ragged hole was stopped with the beveled 'stop waters' which were made use of. The dam was pumped out in about 6 hours and the leakage was so small that a 3-inch discharge pulsometer kept out the water, and was then run only at intervals. Small leaks were stopped by dumping rotten stable manure in their vicinity."

It is interesting to note that the bulkhead stood a pressure of 12 feet of water. Experiments made to determine what pressure 12-ounce duck would stand, show that the clean canvas begins to leak at 2 pounds pressure, and at 5 pounds pressure the leakage becomes a marked amount. With mud on the canvas the leakage becomes noticeable at from 5 to 7 pounds, and of a considerable amount at 50 pounds pressure, these pressures being on a circle $4\frac{1}{2}$ inches in diameter. The canvas did not rupture at 800 pounds.

The suggestion is made to use an inverted funnel of canvas to stop the leakage of springs on rock bottom. (Fig. 26.) The canvas to be spread out over the bottom and weighted down with concrete, and the top wired to a pipe into which the water may rise until the pressure-head is overcome or the pipe can be plugged. Arrangements of this nature, but without the canvas funnel, have been frequently used. An iron pipe set on end is fitted over the leak, and after concreting around to make it water-tight, the water rises inside until the pressure is balanced. A water-tight wooden box may also be used for the same purpose.

The founding of a new inlet tower in the Mississippi at the St. Louis water-works was accomplished by using a coffer-dam, and it was the intention to form a junction with the bottom by using a canvas curtain. When the coffer-dam was floated into position and the divers were sent down to spread the canvas and weight it down with stones, it was found to be damaged so as to be useless. This was supposed to be due to the action of the swift current, but was most probably due to some accident such as fouling on a snag or against a barge.

The anchoring of the crib for this dam is related in the *Engineering News* of July 4, 1891. The dam was to be located near the head of a stone dike about 20 feet in height and on solid rock bottom which was uneven and worn into grooves by the action of the current,

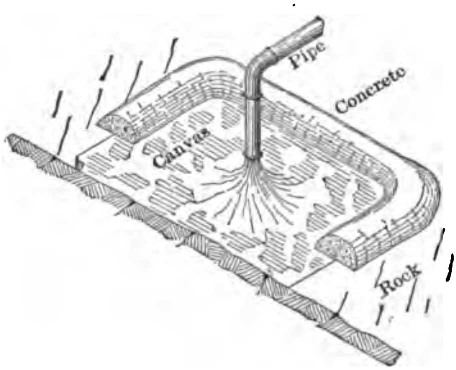


Fig. 26.—Canvas Funnel for Closing Leaks.

which had a velocity of between 6 and 8 miles per hour. The bottom was leveled off by blasting, to receive the crib, which was to be sunk in from 15 to 18 feet of water.

The three triangular cribs shown (Fig. 27) were sunk ana filled with stone and were used to hold the dam in place while building and while being sunk. Steel cables $1\frac{1}{2}$ inches in diameter were used as anchors.

The large crib also served as a protection from the current and drift.

The size of the crib was 38×74 feet outside and the height 22 feet. The 12×12-inch yellow pine timbers were drift-bolted together with from 1 to 2 feet spacing of bolts, and all the joints between the timbers were calked. The bracing consisted of 12-inch square timbers, of which there were three rows, the braces in each row being 4 feet

apart vertically. These were cut out as the masonry was built up and bracing against the stone work substituted.

There were four sets of diagonal bracing as shown. The space between the walls, which was 3 feet, was partly filled with concrete in sacks, and puddle placed on top. Sacks of clay were banked up around the outside, and then the dam was pumped dry with a 10-inch pump. Inside was found 8 feet of mud and 60 sacks of concrete which had been washed there by the swift current.

The amount of timber used was 125,000 feet, B.M., and about 12,000 feet of $\frac{7}{8}$-inch round iron for drift-bolts. The puddle-chamber required 1000 sacks of concrete and 100 barge loads of clay,

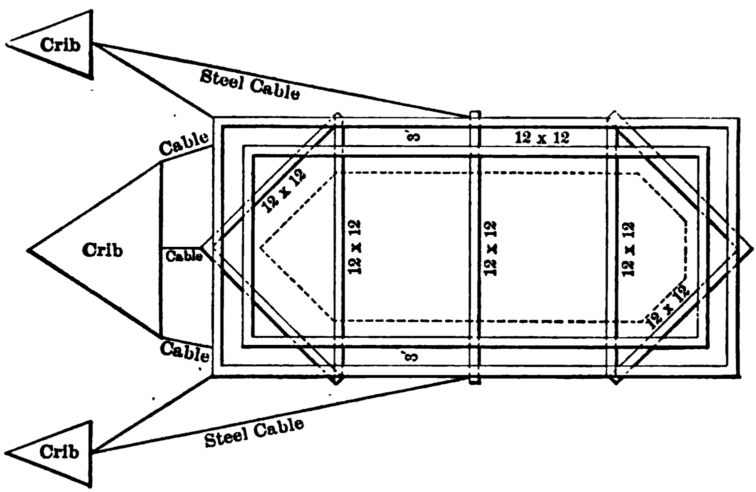


FIG. 27.—CRIBS FOR ANCHORING ST. LOUIS COFFER-DAM.

while 10,000 sacks were used for banking up clay on the outside. This work was constructed under the direction of C. V. Mersereau, Division Engineer, under S. B. Russell, Principal Assistant Engineer.

The Queen's Bridge at Melbourne, Australia, is a plate-girder structure with four piers of 8 cylinders each. The bottom was a reef of bluestone which had been shattered by blasting and which was silted over with about 3 feet of very soft silt.

The use of ordinary puddle coffer-dams was thought to be too expensive, as the bridge was 100 feet in width, and it was proposed to use a single wall of timber protected by tarpaulins. The account of this work is taken from the *Engineering News* of April 4, 1895, which is an abstract of a paper by W. R. Renwick, engineer in charge.

To insure as light a construction as possible experiments were

made on the strength of Oregon pine, and it was found that tests of water-soaked timber showed a loss of strength of as much as 33 per cent., when compared with tests of seasoned timber. The break, too, of the water-soaked pieces was very short. This strength being the one adopted, a very low factor of safety was used. A separate dam was constructed around each tube, but with one side to open as a door to allow its removal and use for another place. The frame was made from 12×12 Oregon pine, with the sticks placed closer together near the bottom to resist the greater water pressure, and 12×12 pieces were run up the corners, the frames being notched in. These also served as spacers for the side timbers and as door frames. The sheeting on the outside was of 4×12 rough timber, and outside of this at the top and bottom were wale-pieces, 6×12, bolted through the frames with 1-inch bolts to hold the sheeting in place.

The tarpaulin was passed completely around the dam, being tacked to the waling-pieces, and so arranged as to allow the door to open.

When the dam had been placed around a tube the sheeting was driven down to rock, through puddle which had been dumped on the bottom, and the pumping was readily done with pulsometer pumps. The only serious leaking was where the 1-inch bolts passed through the joints between the sheeting, but these were plugged with soft wood plugs, and in other work the bolts were flattened to three-eighths of an inch where they passed between the plank. The dams were removed by first drawing the sheeting up to its original position, when the door was opened and the crib taken to another tube. The depth of water was about 15 feet, but while this was successful in this instance, the method should not be copied unless the conditions are favorable, nor unless the cribs are made practically water-tight in themselves.

This was the case in the above work, as one of the tarpaulins was accidentally torn off and the dam still excluded the water, so that the tarpaulin was only a wise precaution. Why the cylinders were not made water-tight and used as their own coffer-dam is not stated, but this possibly could have been done.

Sheets of tarpaulin in closing accidental leaks could doubtless be employed frequently, but as the sole dependence for producing tightness it should be used with extreme care, in a gentle current and well protected from damage.

The pivot pier of the Harlem Ship Canal bridge was founded in a polygonal coffer-dam, from the plans of William H. Burr, consulting

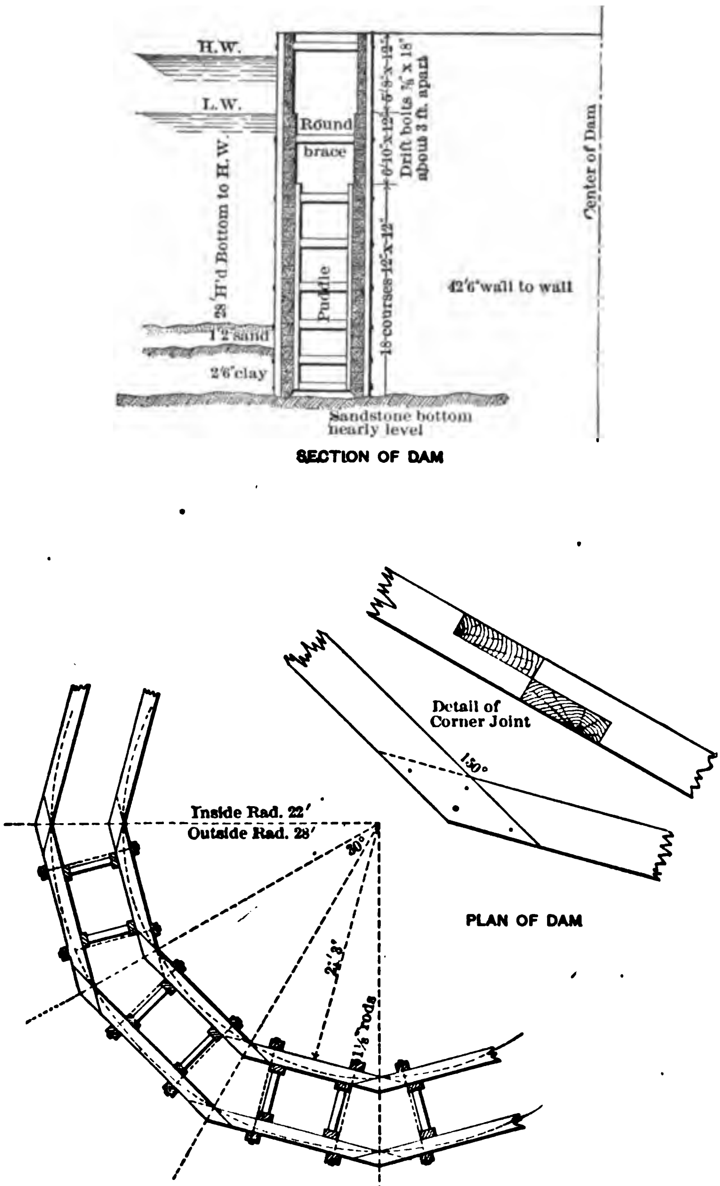


FIG. 28.—DETAILS OF COFFER-DAM USED ON ARTHUR KILL DRAWBRIDGE

engineer. The work is described in the *Engineering Record* of July 24, 1897: "The rock bottom secured by the canal excavation being an acceptable surface for the masonry of the pivot pier, it was constructed in a polygonal double-walled coffer-dam with thirteen sides 25 feet high and 60 feet in extreme diameter. The great dimensions of the coffer-dam would have made it difficult to build and launch it on shore. Consequently it was built partly on a detachable raft. As shown in the illustration (Fig. 29) the inside wall was built up of timbers lapped and halved at the angles; the outer wall timbers were carefully butt-jointed and secured by cross-struts and 1-inch bolts to the inside walls. The rough-sawed horizontal surfaces of the inner wall were bedded in stiff grease and the joints calked, which notably resisted the penetration of the water. Each course of timber was secured to the one below it by $\frac{3}{4}$-inch drift-bolts spaced about 4 feet apart. When the bottom was thoroughly cleaned the concrete was dumped in place by a special steel bucket. Concreting was carried on night and day and was completed before puddling was begun. Considerable difficulty was occasioned by the irregularities of the bottom which the coffer-dam could not be made to fit closely. Divers were sent down and filled in bags of sand, as at *S*, and riprap *R* was piled up outside to protect it. Then the space between the walls was filled with puddle."

Another polygonal dam was constructed for the draw pier of the Arthur Kill bridge, by Alfred P. Boller, consulting engineer. The following account is taken from Vol. 27 of the Transactions Am. Soc. C. E.: "It was necessary to use as little space as possible for the dam, and to construct it without interior bracing, so that a double-walled twelve-sided polygon (Fig. 28) with walls 4 feet apart in the clear was used. The rock bottom was over-laid with 2 feet of clay and the clay with 18 inches of sand and mud, the depth of water over the rock being 28 feet at high tide. The square hemlock timbers used in the walls were halved together and the walls braced together by bolts and round timbers for struts, the round timbers allowing the puddle to run around them and pack well as thrown in. Clamp timbers 4×6, in two lengths, were held in place by the bolts and the struts were braced against 6-inch plank. The dam was built to one-third its height on shore, then towed to position and built up until grounded. Between the timbers and the joints candle-wicking was placed, and the courses drift-bolted together every 3 feet and spiked at the joints. The rock was dredged bare before placing the crib, which was filled with a hard, gravelly clay between the walls after being sunk in place. A rich Portland concrete was dumped inside,

from triangular buckets, to seal the bottom, and then the dam was pumped out with a 6-inch pump and kept dry by pumping at intervals. In one place the concrete was not thick enough and a spring came up through a fissure in the rock. This was boxed in and led to the sump. The material used was 140,000 feet of timber, 15,000 pounds of iron, and 600 yards of puddle."

A piece of work similar to the Canadian Pacific example was an octagonal single-walled dam used in the construction of the Coteau

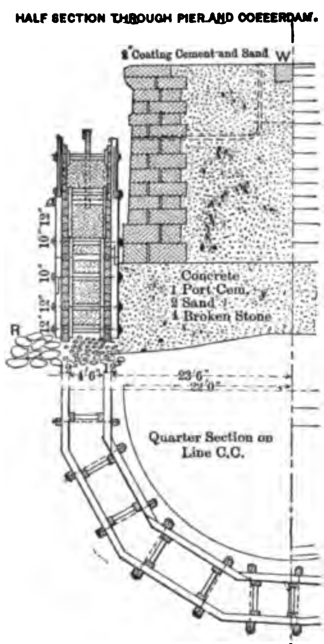


FIG. 29.—POLYGONAL COFFER-DAM, HARLEM SHIP-CANAL DRAWBRIDGE.

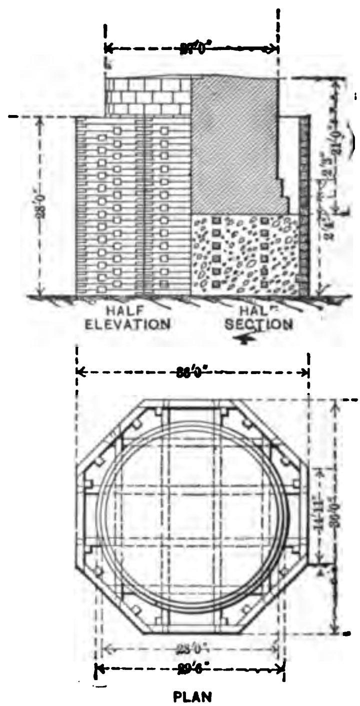


FIG. 30.—COFFER-DAM FOR PIVOT PIER OF THE COTEAU BRIDGE.

bridge on the Canada Atlantic Railway. This is illustrated in the *Engineering News* of May 30, 1891 (Fig. 30). The dam was braced thoroughly with cross-timbers built into the sides, and the bottom being of rock it was partly filled with concrete to make it water-tight.

The following account of a very cheap and novel method of construction is from the *Engineering Record*, Aug. 2, 1913:

"Chelsea Bridge North carries highway and street-car traffic

over the north or main channel of the Mystic River, between the Charlestown District (Boston) and Chelsea, and forms a part of one of the most important highways out of Boston. It has pile approaches, built in 1880, and had a retractile drawspan built in 1895 and lengthened in 1900 to span a waterway 60 ft. in width.

"On account of the rapidly increasing water traffic of the Mystic River, and because the waterway was in an unsatisfactory location, the Secretary of War, on January 3, 1910, ruled that the bridge was an unreasonable obstruction to the free navigation of the river and ordered that the clear width of the draw opening be increased to 100 ft. or more.

"The new drawspan is to afford two waterways, each 125 ft. wide in the clear and 30 ft. deep below low water. It will be 363 ft. long over all, 60 ft. wide, and will weigh 1400 tons, affording a clearance above mean low water of 25 ft. The contract for building the permanent pivot pier and the wooden fender piers, for the rebuilding of the existing pile approaches, and for building and removing the temporary by-pass bridge to provide for travel during the reconstruction was awarded in February, 1912, to Mr. George T. Rendle, of Boston, and will cost approximately $200,000. The estimated total cost of the whole work, including the draw superstructure, is $425,000.

"The pivot pier has a concrete base 60 ft. in diameter and 39 ft. high, with its foundations on solid rock and the upper part about 1 ft. above mean low water. The weight of the base and the volume of concrete required for it are reduced by the construction of a concentric cylindrical chamber 30 ft. in diameter and 22 ft. high in the upper part. The top of this chamber is spanned by steel I-beams supporting part of the load from the pier shaft above, which is 51 ft. in diameter and about 13 ft. high from the top of the base to the top of the coping. It is faced with five courses and a coping of quarry faced granite, backed by a solid mass of concrete, which, like that of the base portion, is made of 1 : 2 : 4 Edison Portland cement, sand, and stone up to 2 in. in diameter.

"At the pier site an excavation 65 ft. in diameter was dredged to bedrock, which was found at depths varying from 30 to 38 ft. below low water. The material was removed chiefly by a dipper dredge and consisted of about 8 ft. of soft silt mixed with sand, from about 17 ft. to 25 ft. below low-water level, beneath which there were about 8 ft. of blue clay, and then a stratum of very hard sand, gravel and clay, with a thin stratum of shale covering the bedrock. The slopes of the excavated area were maintained at 2½ horizontal to 1 vertical.

"The basket crib, Fig. 31, or form for the pier foundation, was built of about one hundred and forty-five horizontal courses of 3×12-in. yellow-pine planks, 8 ft. long, laid flat and breaking joints. The ends were beveled to make radial joints, and each plank was secured to those below it by 1-in. oak treenails 9 in. long, two at each end of each plank, about seven thousand eight hundred treenails being required for the entire crib. In addition the planks were well spiked to the lower courses throughout their entire length with 6-in. spikes. The courses were also secured together by 4×12-in. vertical planks opposite alternate joints, which were fastened to the inner circles of the crib by lag screws. The crib, which contained about 83,000

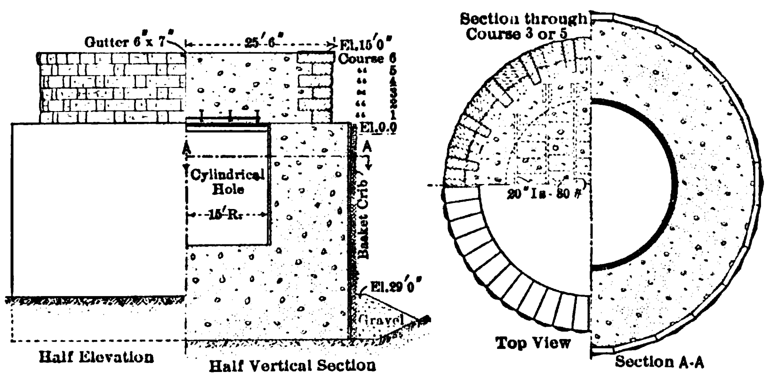


FIG. 31.— COFFER-DAM FOR PIVOT PIER, CHELSEA BRIDGE.

ft. B. M. of yellow-pine plank, was built up to a height of 3 or 4 ft. on shore between high- and low-water marks, and was then floated to deeper water and completed while still floating. After completion it was towed into position and held by guide piles, spaced about 12 ft. apart around its circumference.

"To sink the crib it was first planned to build exterior pockets which would be filled with gravel, but they were eventually dispensed with, and the crib was sunk by loading it with old iron, with stone intended for use in the pier masonry, and with heavy chains hung over the walls of the crib.

"There was no attempt to construct the crib so that on the bottom it should conform to the variations of the rock surface. Instead, the bottom of the crib was made level and it was sunk until it took bearing on only a portion of the lower edge at the highest rock level. Then, to provide continuous bearing at all parts of the circumference, and especially to complete the inclosure of the crib and confine the

concrete that was afterward deposited within it, wooden boxes of varying size, but averaging about 4 ft. square, and 4 ft. deep, were filled with lean concrete, lowered to the bottom and placed by divers under the edge of the crib to form a continuous wall. After the concrete boxes were placed the excavation outside of the crib was backfilled with gravel and dredged material until the whole crib was surrounded by filling to about 29 ft. below low water, or some 2 ft. above the bottom course of plank of the crib. This back-filling formed an effectual seal to retain the concrete which was deposited in water inside the crib without unwatering the latter.

"Broken stone and sand were delivered both by lighter and by team. The cement was conveyed by team. All the materials were stored on the old bridge close to the draw opening, where the stone and cement were measured by shoveling into scale boxes. Between the old bridge and the pivot pier was moored a concrete mixing scow, equipped with a Milwaukee mixing machine and a boom derrick. The derrick delivered the scale boxes of sand and stone from the old bridge to the charging hopper of the mixing machine. The concrete was mixed in 1-yd. batches and delivered to a submarine bucket with bottom flap doors. The bucket was handled by the derrick on the mixing scow and was so constructed that the doors were not opened until the bucket was seated on the bottom of the pier foundation, after which the concrete was automatically deposited as the bucket was lifted by the derrick.

"After the concrete foundation had been built up to within 20 ft. of low-water level a permanent form was set in the center of the pier for the cylindrical chamber above referred to, and the remainder of the concrete was deposited in a concentric ring between this form and the basket crib. The concrete was deposited in water up to about low-water level, and as the basket crib extended about 5 ft. above low water it was possible to unwater the crib at half tide or less, and the remainder of the work on the pier was carried on in the dry. This consisted in placing 20-in. steel I-beams over the hollow center, the building of forms upon these I-beams, and the covering of the whole pier foundation with a layer of concrete about 2 ft. in depth, which was leveled off in readiness for the laying of the stone masonry of the upper portion of the pier.

"The pier contains about 3673 cu. yds. of concrete and 323 yds. of granite masonry. The construction of the basket crib was commenced Aug. 8, 1912. It was sunk in position Sept. 14, 1912. Concreting was commenced Sept. 28, 1912, and the pier was completed ready to receive the superstructure Dec. 7, 1912.

"The work was designed and executed under the direction of the Public Works Department of the City of Boston, of which Mr. L. K. Rourke is commissioner; Mr. Frederic H. Fay, engineer, Bridge and Ferry Division, and Mr. S. E. Tinkham, engineer of construction."

The different forms of sheet-piling will next be taken up, together with the pile-driving machinery and the methods of driving both sheet- and guide-piles. After this will be described the use of sheet-piles for forming water-tight coffer-dams, by reference to actual constructions of that character.

CHAPTER IV

PILE-DRIVING AND SHEET-PILES *

In no department of engineering have ancient methods been more rigidly adhered to than in that of pile-driving. The form of the pile-driver derrick has remained so characteristic that a person but slightly familiar with the subject would have little difficulty in recognizing the pile-driver in the picture of Cæsar's Bridge (Fig. 3) in Chapter I. The bridge of the Emperor Trajan over the river Danube is an instance of the early use of piles. This bridge was constructed in the first century, and when the piles under water were examined in the eighteenth century they were found in some cases to have become petrified to a depth of three-fourths of an inch from the surface, beyond which the timber was in its original state. Before derricks were used it is probable that piles were driven by a large maul of hard wood, which is termed by Cresy a "three-handed beetle." The block of hard wood was hooped with iron and had two handles radiating from its center, to be worked by two men, while a third man assisted in lifting it by means of a short handle opposite.

Wooden mauls are still used where sheet-piling is to be driven into a soft bottom, and heavy iron mauls or sledges are also used; but as has been frequently stated such a soft bottom should be dredged and some more elaborate apparatus used to drive the piles into a harder substratum.

The most primitive form of the pile-driving derrick is similar to the one used in 1751 by the celebrated French engineer, Perronet, at the bridge of Orleans (Fig. 32). This was arranged with a number of small ropes splayed out from the end of the lead line, so that a number of men could pull down at one time, the drop of the hammer,

* The subject of pile-driving has been restricted to the ordinary methods and operations; such unusual processes as gunpowder pile-driving and the like have not been referred to.

Pile-driving, with the assistance of the water-jet, has been described in Chapter V and in the account of the Sandy Lake coffer-dam. The ordinary operations of pile-driving, as practiced on that work, are also described in some detail.

of course, being limited by the reach of the men's arms. The windlass shown was for the purpose of raising the pile into place between the leads.

The same engineer improved upon this derrick by adding a large bull-wheel to the windlass, on which was wound a rope to be pulled by a horse from the side, as shown in Fig. 33, thus winding up the lead line on the windlass. This same apparatus is in use down to the present time, except that one seen recently had the windlass at right angles to the one illustrated.

The ram or hammer used in olden times was of oak, bound with iron, and weighed for the work at Orleans 1200 pounds for the main piles, which were 9 to 12 inches in diameter and which were driven 3 to 4 feet apart, center to center, to a depth of 6 feet into the bed

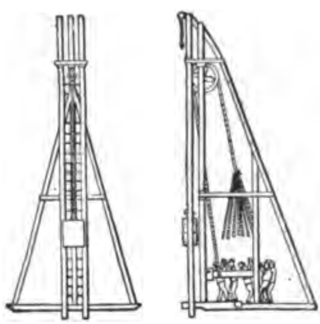


FIG. 32.—PERRONET'S PILE-DRIVER.

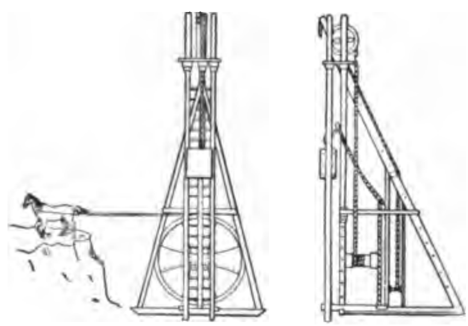


FIG. 33.—PERRONET'S BULL-WHEEL PILE-DRIVER.

of the river; the ram for the sheet-piles only weighed half as much, the sheet-piles being about 12 inches wide by 4 inches thick.

At the bridge of Saumur, which was built about the year 1756, De Cessart employed a driver with a bull-wheel, in the periphery of which were set pins, to form handles for the men to pull upon and rotate the wheel. Eight men, by making three turns of the wheel, raised the ram weighing 1500 pounds 6 feet, when it was unhooked and allowed to drop. The piles cost from two to five dollars each in place.

A very simple form of pile-driver is shown in Fig. 34, and was described in the *Engineering News* of March 16, 1893, by Julian A. Hall. The hammer is hewed out of a section of a hardwood log, and has pieces bolted on the sides to hold it in the leads, which should give plenty of clearance. The derrick was constructed of very light timber, the verticals being 4-inch sawed stuff and the bottom tim-

bers 6×6 inches. The rope passes over the sheave *A* and down over the tops of the steps *B*, *B*, on which the men stand to pull the line and thus operate the hammer. This was a very inexpensive apparatus and was found to work well. Where there is already in use a heavier hammer of cast iron it can be used by striking light blows. The construction of the ordinary pile-driver derrick is a simple piece of framing, when good straight timber is easily obtained, the essential features being to keep the leads free from any obstruction for the hammer and to have efficient bracing.

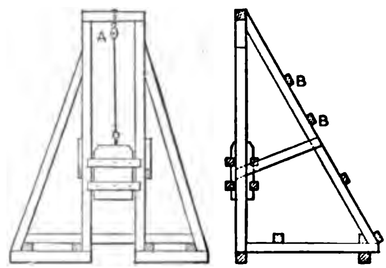


FIG. 34.—SHEET-PILE DRIVER.

For bracing a derrick under 25 feet a straight-back brace or ladder having two horizontals running to the leads, and two side-braces will be sufficient. But for a higher one, either additional long braces should be used or diagonals introduced between the leads and the ladder. The use of long braces is shown in Fig. 35, which is the design of pile-driver such as is used about harbors or rivers on heavy work. It would be mounted on a scow or flatboat 60 feet in length, 25 feet in width and of about 6 feet in depth. The design of smaller derricks can be approximated from this one, the bracing being used in proportion.

It will be noticed that the guides for the hammer are 4″×4″ lined with a steel plate. Two lines are provided, one being for the operation of the hammer and the other for pulling piles into place. Especial attention is called to the hooks at *A*, as these are seldom shown in the plan of a derrick and they are of constant use for clamping and guiding piles. A timber laid across is wedged tight against the pile to draw it to line, and can be used to correct a stick which is beginning to slant badly. Similar clamps of course are used on the opposite side of the leads.

Where a pile begins to sliver or split in driving, if the sliver is spiked down and the clamps used to hold it in place, the trouble can usually be corrected before the pile is badly damaged.

The use of diagonal bracing between the leads and ladder is shown in the Lidgerwood derrick (Fig. 36) in which a diagonal is introduced between each pair of horizontals. This form of bracing is very satisfactory and equally as good as the other method. The diagonals on a very large driver may be extended over two panels

and planks spiked down to the horizontals to form a platform for the workmen. In smaller derricks the diagonal bracing is most always omitted, dependence being placed in the stiffness of the leads and the bracing from the ladder and horizontals, as was done in the derrick shown in Fig. 4.

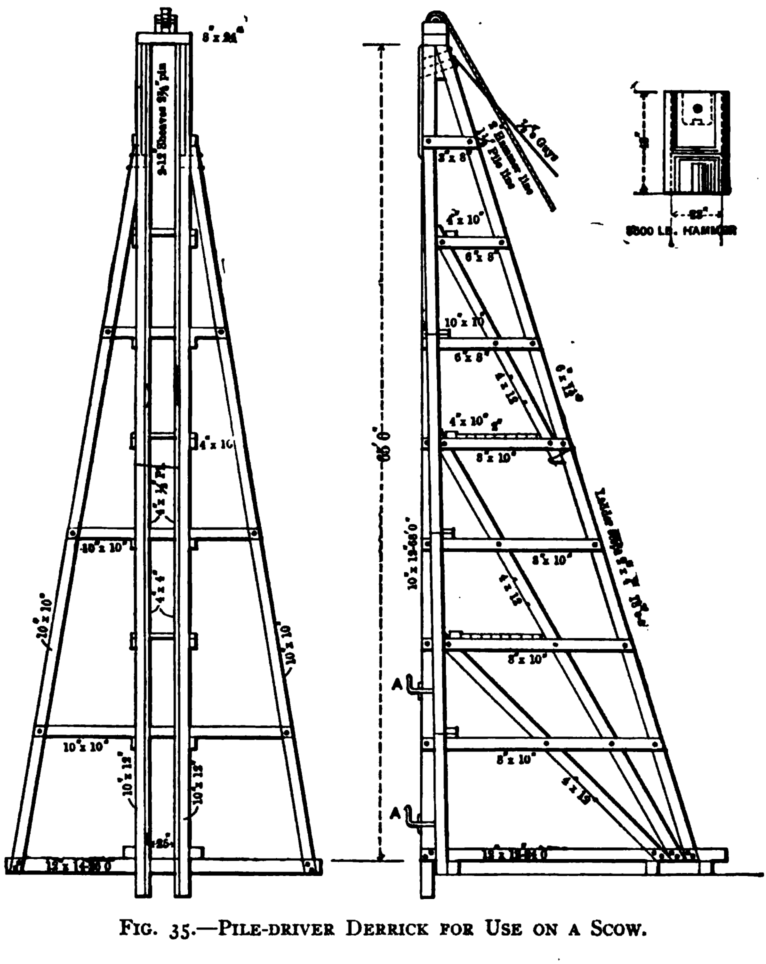


FIG. 35.—PILE-DRIVER DERRICK FOR USE ON A SCOW.

The power for driving with a small hammer weighing from 500 to 1500 pounds, may be furnished by laborers pulling, but this is a slow operation and horse-power is nearly always used where steam is not available. The power is furnished from a drum with a long lever, to which the horse is hitched and winds up the hammer by walking in a circle about the drum, the frame of which is firmly fast-

ened in place. This is called a "horse-power" apparatus and works slowly, but is a cheap and satisfactory way where a very few piles are to be driven. To the hammer-line are attached the tongs or nippers, which engage the pin in the top of the hammer (Fig. 37), and when the hammer has reached the proper height it is dropped by pulling a tripping-rope and releasing the tongs, or if the hammer is hoisted to the top of the leads, the top arms of the tongs are pushed together by the wedges on the leads and the hammer released automatically. This is a slow method on account of waiting until the tongs run down again and engage the hammer. The horse-power, of course, has a ratchet, so that the rope runs down free and usually the

FIG. 36.—LIDGERWOOD PILE-DRIVING DERRICK.

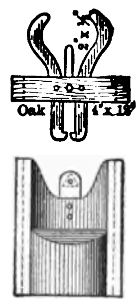


FIG. 37.—HAMMER WITH NIPPERS.

blows are hurried by overhauling the line. With the addition of a hoisting-engine all this is changed and pile-driving becomes one of the most stirring operations of the contractor. When the hammer is hoisted, the friction lever is released and the hammer descends, carrying the rope with it, as the tongs are done away with and the line attached directly to the hammer. A good engine man will catch the hammer on the rebound and materially lessen the time between the blows and likewise the cost of driving.

With a heavy hammer shorter drops are made, thus causing much less damage to the pile, which would split badly under the high drop from the use of tongs. For the smaller-sized hammers —from 1000 to 1500 pounds—an engine of 10 horse-power is mostly used, as it is usually thought best to have a surplus of power in case of need; while for a 3000-pound hammer a 20-horse-power engine

would likely prove the best and most economical, but not infrequently a 25-horse-power hoist is employed.

The cost of an outfit will vary greatly and the only satisfactory way is to get prices from responsible firms, but for preliminary estimates the cost of a 10-horse-power hoist with single cylinder and single drum may be taken at about $900, and for a 20-horse-power at $1270. Preliminary prices for other sizes of single-cylinder, single-drum hoists, may be obtained from the formula:

$$\text{Cost} = \sqrt{81{,}000 \times \text{horse-power}}.$$

The double-cylinder engines will cost about 10 per cent. more and double drums about 10 per cent. additional to this.

Pile-driver derricks will vary much in cost owing to the location, on account of the cost of timber, but a minimum cost for a first-class derrick will be $6 per vertical foot and a maximum of $8. Being such a simple structure the easiest and safest way will be to make an estimate for each case.

In the selection of an engine it is well to remember that with a double drum a second pile may be hoisted into place, while the first one is being driven, as all derricks are, or should be, provided with two sheave wheels at the top for this purpose. While a single-drum engine has a spool for this purpose, it cannot be used very satisfactorily.

A pile-driver on a scow is shown in Fig. 38, such as was used in driving piles on the New York State canals. Another pile is just being hoisted into position. The hoisting-engine has no protection, but a shed or house is mostly provided as a protection from the weather.

While little change has ever been effected in the design of pile-driving derricks, the adoption of steam-hoists was a great improvement, as was also the invention of the steam pile-hammer by James Nasmyth. The principle is the same as that of steam forging-hammers, and was applied by Nasmyth to pile-driving in 1845, the hammers of this class bearing his name to-day. His idea was that the drop-hammer was calculated more for destruction than for useful effect and he termed it the "artillery or cannon-ball principle." Besides this the action of the drop-hammer even with the use of the "monkey" engine was somewhat slow.

Samuel Smiles says that "in Nasmyth's new and beautiful machine he applied the elastic force of steam in raising the ram or driving-block, on which, the driving-block being disengaged, its

whole weight of three tons descended on the head of the pile, and the process being repeated eighty times in a minute the pile was sent home with a rapidity that was quite marvelous as compared with the old method. In forming coffer-dams for piers and abutments of bridges, quays, and harbors, and in piling the foundations of all kinds of masonry the steam pile-driver was found of invaluable use by the engineer. At the first experiment made with the machine Mr. Nasmyth drove a 14-inch pile 15 feet into hard ground at the rate of sixty-five blows per minute. The saving of time effected

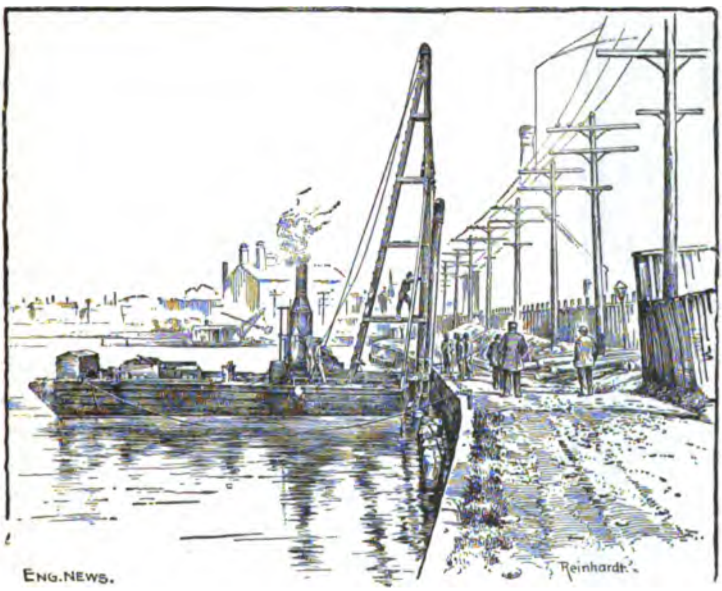


FIG. 38.—PILE-DRIVING SCOW, NEW YORK STATE CANALS.

by this machine was very remarkable, the ratio being as 1 to 1800; that is, a pile could be driven in four minutes that had before required a day. One of the peculiar features of the invention was that of employing the pile itself as the support of the steam-hammer part of the apparatus while it was being driven, so that the pile had the percussive force of the dead weight of the hammer as well as the lively blows to induce it to sink into the ground. One of the most ingenious contrivances of the pile-driver was the use of steam as a buffer in the upper part of the cylinder, which had the effect of a recoil spring and greatly enhanced the effect of the downward blow."

Many modifications of this hammer have been manufactured, and one much used at present is the Warrington-Nasmyth hammer, made by the Vulcan Iron Works. This hammer (Fig. 39) is made in three sizes, the weight of the striking parts being 550 pounds for sheet-pile work, 3000 pounds for medium pile work, and 4800 pounds for use on heavy work. This machine is provided with a positive valve-gear, a short steam passage to avoid the waste of steam, a wide exhaust opening to prevent back pressure as the hammer drops, a piston-head forged on the rod, and channel bars on the side to allow the pile to be driven lower than the leads of the derrick. The hammer is attached to the hoist rope, but this is left slack when the hammer is resting on the head of the pile, steam is turned on and the hammer pounds automatically at the rate of sixty to seventy blows per minute until the pile is driven. The bottom casting which rests on the pile is a bonnet which encases the top and prevents brooming or splitting.

FIG. 39.—WARRINGTON-NASMYTH STEAM PILE-HAMMER.

The hammer should have plenty of play in the leads, and the steam-pipe should extend half way up the derrick to save length of hose. This hammer has a record of as high as seventy-five to one hundred piles per day, and one account gives the record of 3000 lineal feet of piling per day at a cost of $50, the number of men employed being sixteen and the coal consumption one ton. This hammer is shown in Fig. 40 in use driving piles for bridge work on the Fair Haven bridge.

Another form of the Nasmyth hammer is the Cram (Fig. 41) which is very simple in construction. The driving-head is hollow and the steam enters through a hollow piston-rod, causing the head or cylinder to rise on the rod. Four sizes are made, with hammers of 430 pounds, 2000 pounds, 3000 pounds, and 5500 pounds. The small hammer, which is listed at $300, is used for sheet-pile work by inserting a "follower" of oak which fits the base or pile cap, and which has a slit in the lower end to fit the sheet-pile. The number of

blows per minute is the same as other steam pile-hammers and an average of eighty-three piles per day of ten hours is reported, where they were driven 17 feet into sand and oyster-shells in the Passaic River, the largest day's work being 121 piles, or nearly double the best work with an ordinary hammer.

There are a number of new types of steam pile-hammers on the market, one of the best of them being the Arnott, manufactured by

FIG. 40.—WARRINGTON-NASMYTH HAMMER, FAIR HAVEN BRIDGE.

the Union Iron Works of Hoboken, N. J. As may be seen from Table I, either steam or compressed air can be used, as may be most convenient, but the size boiler given would be too small if jetting pumps are to be used; the amount of horse-power of boilers to be added for this may be found in Chapter V on Jetting Piles. It must be borne in mind, however, that a steam hammer will keep the pile moving all the time and in many cases accomplish the same results as jetting, and make it unnecessary to provide a jetting plant.

TABLE I.—ARNOTT STEAM HAMMERS.

Size Number.	Total Weight. Pounds.	Weight of Ram. Pounds.	Dimensions Over All. Height × Width × Depth. Inches.	Cylinder. Diam. × Stroke. Inches.	Total Downward Force Steam Plus Ram Pounds.	Number of Strokes per Minute.	Power in Foot-pounds per Minute.	Steam Boiler H.P. Req. at 80 lbs. Pressure per Sq. Inch.	Comp. Free Air per Minute at 80 lbs. Pressure per Sq. Inch Cu. Feet.	Suitable for	Size of Hose Inches.
0	12,100	2,550	118×28×20	$10\frac{1}{2}$×24	7,800	100	1,562,000	50	750	Large concrete piles. steel pipe and sheeting, extra large round and squared piles and sheeting.	2
1	8,000	1,548	94×28×18	$9\frac{1}{2}$×21	5,800	110	1,117,000	30	600	Medium concrete piles, heavy steel sheeting and pipe, large round and squared wood piles and sheeting.	$1\frac{1}{2}$
2	5.500	890	81×25×15	$7\frac{1}{4}$×16	3,300	130	577,700	18	300	General work, small concrete piles, medium round or squared wood piles and sheeting, ordinary steel sheeting.	$1\frac{1}{4}$
3	4,500	663	74×23×13	$6\frac{1}{4}$×14	2,470	135	445,200	15	200	Minor work, small round or squared wood piles, 3″ to 6″ wood sheeting and light steel sheeting.	$1\frac{1}{4}$
4	2,500	363	60×20×11	$5\frac{1}{4}$×12	1,683	150	252,450	10	150	Light work, light round or squared wood piles, 2″ to 6″ wood sheeting and light steel sheeting.	1
5	1,400	214	47×17×9	$4\frac{1}{4}$×9	1,085	200	162,750	8	100	2″ to 4″ wood sheeting and very light steel sheeting.	1
6	850	129	40×14×8	$3\frac{1}{4}$×7	636	250	93,180	5	60	2″ to 3″ wood sheeting.	$\frac{3}{4}$
7	365	70	31×10×6	$2\frac{3}{8}$×5	364	300	45,000	3	40	1″ and 2″ wood sheeting.	$\frac{3}{4}$

Efficiency is computed on 60 lbs. mean effective pressure at cylinder.

The Arnott hammer uses the pressure of the steam in striking the blow and on account of the large number of blows per minute,

FIG. 41.—CRAM-NASMYTH STEAM PILE-HAMMER.

the result of the use of this hammer is greater in foot pounds per minute than almost any other hammer on the market.

The hammer, fitted with a wood-cushioned head for driving concrete piles, is shown in Fig. 42, and for such use it should be

either the No. 0 or the No. 1 hammer. The No. 1 hammer is also the best for general driving, although for smaller wood piles the No. 2 hammer will give good results.

FIG. 42.—ARNOTT-NASMYTH STEAM HAMMER.

The guides for the leads are shown in Fig. 43, although the dimensions given in Table II may be slightly varied.

"All standard stock hammers are provided with heavy angle iron guides, attached with hexagon head tap bolts as per data below. One of the many advantages of detachable guides is that the hammer can be placed in standing leads by removing the rear guide tap bolts, backing hammer in place and replacing guides, thus obviating the necessity of digging pits, etc., for entering the hammer if they are not removable.

"Note that dimension A is actual width of hammers and cannot be altered, but space B between angles can be slightly increased as far as permissible and can be decreased as may be desired.

"Unless otherwise directed guides will be attached as per B."

The floating drivers used by the author have usually been on scows $20 \times 60 \times 4$ feet deep, so that they would be of use on bents closely spaced or in confined locations, and sometimes they have been as narrow as 18 feet. The depth must

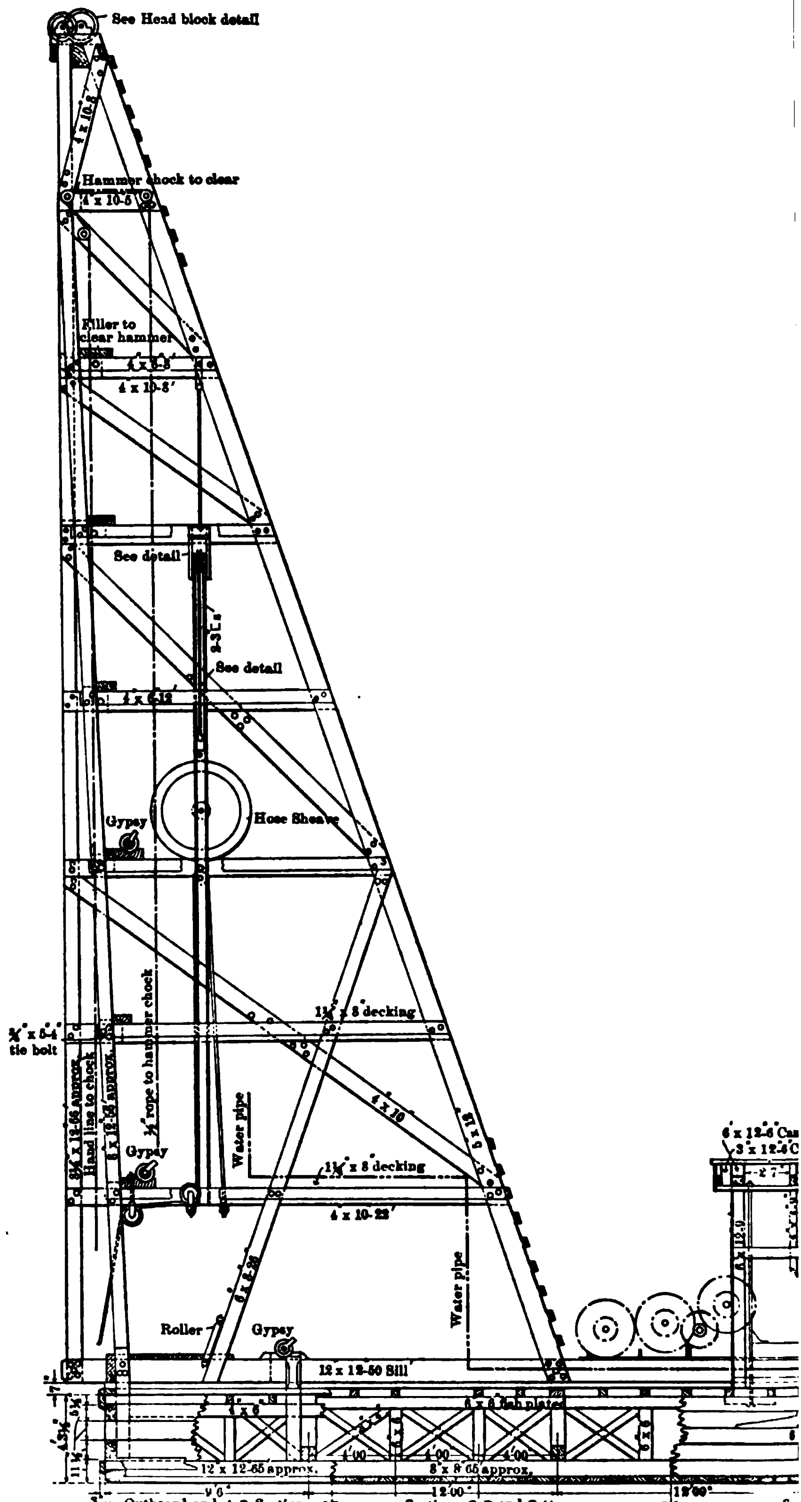
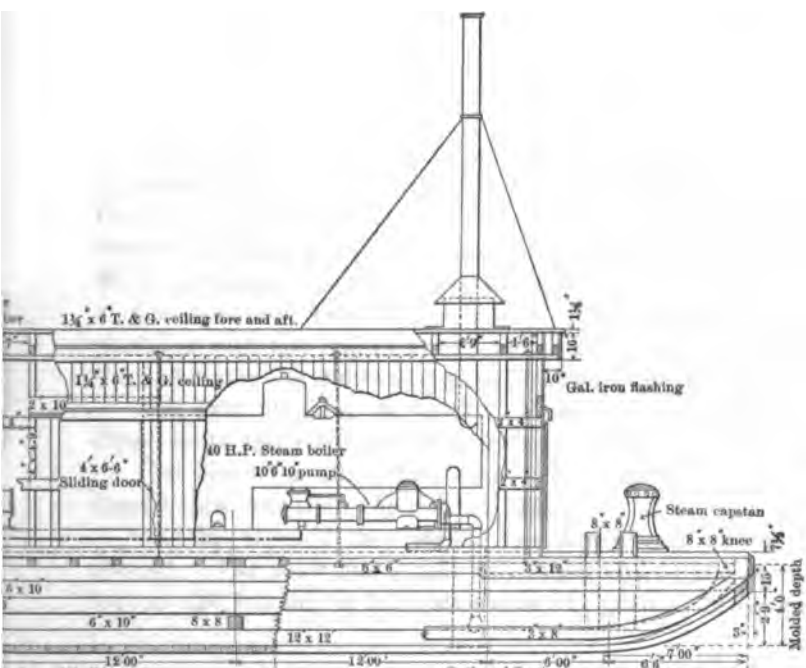


Fig. 44.—United Stat



FLOATING PILE DRIVER.

(To face page 67.)

usually be kept down to 4 feet to allow of use in shallow water, and for working close in on beaches on the tides.

TABLE II.—ARNOTT STEAM HAMMER GUIDES.

Size.	0	1	2	3	4	5	6	7
Dimension A.......	28″	28″	25″	23″	20″	17″	14″	10″
Dimension B.......	$8\frac{1}{2}$″	$8\frac{1}{2}$″	$6\frac{1}{2}$″	$5\frac{1}{2}$″	$4\frac{1}{2}$″	$4\frac{1}{2}$″	$3\frac{3}{8}$″	$3\frac{3}{8}$″
Dimension C.......	8″	8″	6″	5″	4″	4″	3″	3″

The complete plans of a floating driver recently constructed at Portland, Oregon, under J. F. McIndoe, Major, Corps of Engineers, U. S. A., for use on Government work, Figs. 44, 45, 46, 47, and 48, show a first-class piece of plant of this character.

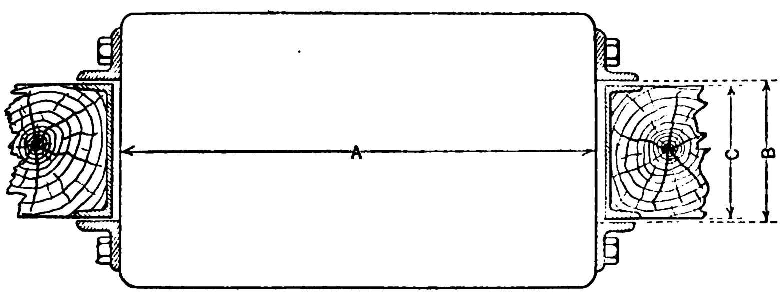


FIG. 43.—ARNOTT STEAM HAMMER GUIDES.

The scow is somewhat larger than ordinary, 24×70 feet $\times 4$ feet $7\frac{1}{2}$ inches deep, and is not provided with a fresh-water tank built into the hull, as is necessary for a driver to be used on salt water. The bottom planking is 4×10 inches and the deck planking $3\frac{1}{2} \times 6$ inches. Wide planking is usually fastened with three $\frac{3}{8} \times 7$-inch boat spikes at the ends and two boat spikes at intermediate points. Narrow planking with two spikes at the ends and one at intermediate points. For salt water the spikes and all other fastenings should be galvanized.

The side planking or strakes should be through-fastened with clinch-bolts and rings and edge-fastened with drift-bolts.

The holes for all spikes and drift-bolts to be bored one-sixteenth inch less than the bolts or spikes.

On the outside they should be countersunk and filled in with cement. The scow has a rake or sloping end at the stern of 7 feet, and all the framing is fully shown on the plans. Scows for the best

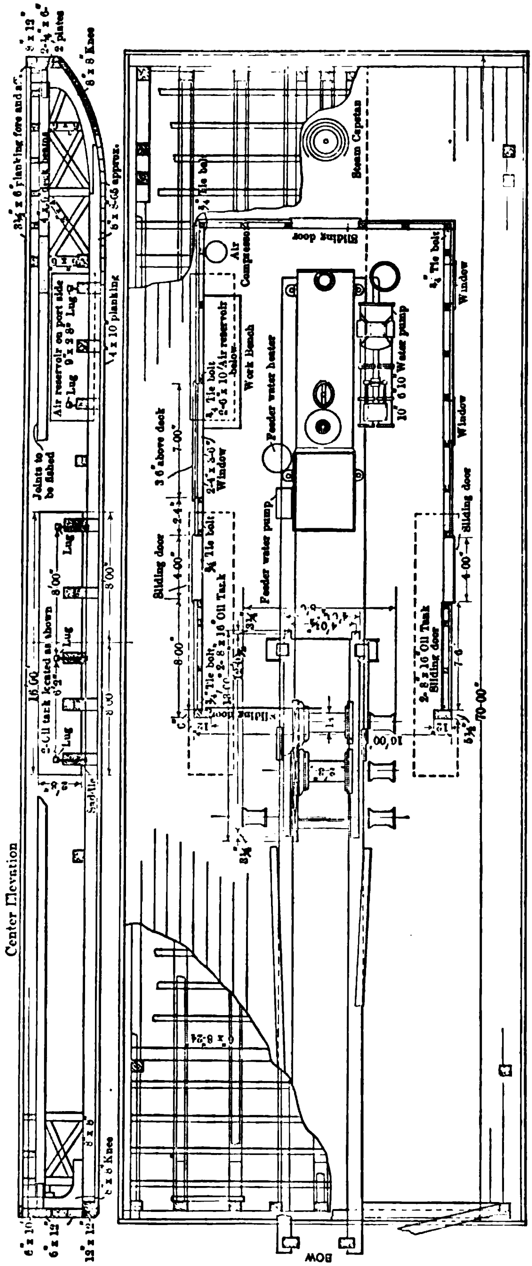


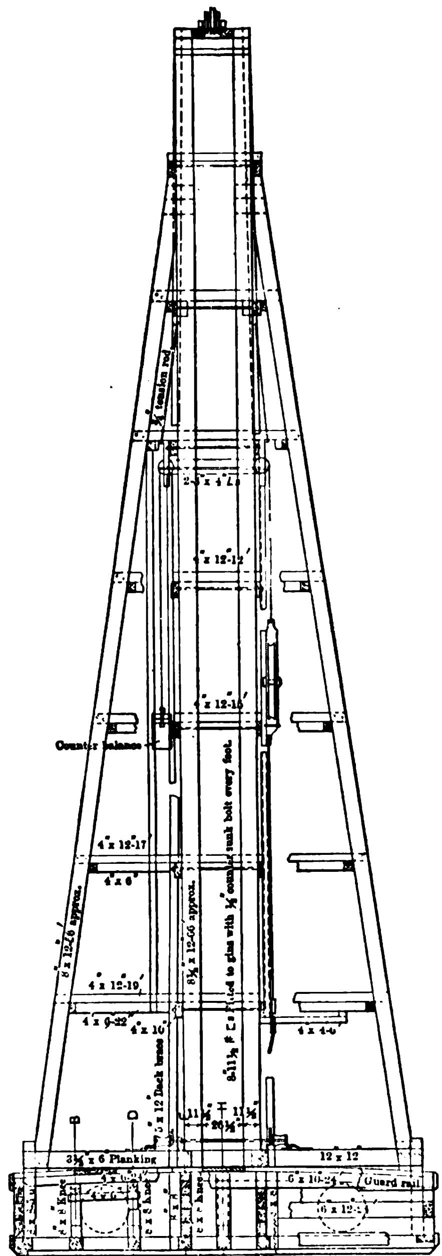
FIG. 45.—PLAN OF U. S. FLOATING DRIVER.

of work should be calked on the deck and sides with three threads of oakum, well hawsed in, and the bottom with four threads of oakum.

The seams below the water-line should all be filled with cement, after calking, and those above the water-line filled with pitch. For light or temporary work two threads of oakum may be used on the deck or sides and three threads on the bottom.

The scow is stiffened with two inside longitudinal bulkheads 6 inches thick, running full length. The leads are 66 feet high, of $8\frac{1}{2} \times 12$-inch timbers, lined with 8-inch $11\frac{1}{4}$-pound steel channels for the hammer guides. The ladder or back braces are 5×12 inches, with steps spiked on as shown. The inside braces are 6×8 inches and 4×10 inches.

Where it is necessary to tip the leads forward for driving batter piles, guys should run from the top of the leads, as shown in Fig. 35, to the stern of the scow. The head block (Fig. 48) has three sheaves, one 18 inches for the hammer line and two 16 inches for pile lines. The author's practice is to use four sheaves on the head block,



AFTER END VIEW

Fig. 46.—Leads, U. S. Driver.

one for the hammer line, one for the pile line, and two outside ones for handling jets.

Next in importance to the scow and leads being of good design is the necessity for a first-class boiler and engine. The boiler shown in Fig. 44 is a horizontal locomotive type of 40-horse-power, or large enough to handle the jet pump for light work, in addition to operating the drop-hammer, or large enough to operate a steam pile-hammer, but not large enough to handle a hammer and two jets for heavy jetting. The usual engine for a floating-driver is a double 7×10, with the ordinary vertical boiler attached. The one shown on the plan, a double $8\frac{1}{2}\times 10$, is provided with two spools or nigger-heads on a separate shaft to handle the bow lines coming over the gypsies. The stern lines are handled on the steam capstan shown. This driver is provided with two tanks below deck for fuel oil in

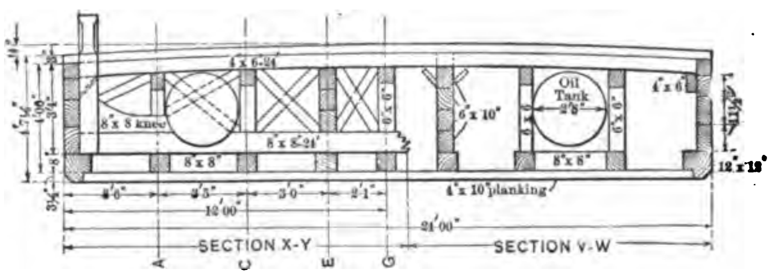


Fig. 47.—Section, U. S. Driver.

place of coal, and an air receiver to be supplied with compressed air from the air pump, for operating the wood-boring tools or such other air plant as may be found needful.

The jetting pump is a $10\times 6\times 10$ duplex center-packed pump, which will deliver from 200 to 220 gallons of water per minute at a pressure of not less than 200 pounds per square inch.

The other equipment is the feed-water pump, the feed-water heater, and the fuel-oil pump. The hoist engine shown, a double-drum, double $8\frac{1}{2}\times 10$, would be heavy enough for a large steam-hammer and for piles a hundred feet or more in length.

The scow should be protected originally by two coats of copper paint below the water line, the bottom covered with tarred ship felt and 2-inch plank sheathing, which should also have two coats of copper paint. For protection against teredo, this copper paint should be renewed about every six months. The remainder of the woodwork should be painted with two coats of the best lead

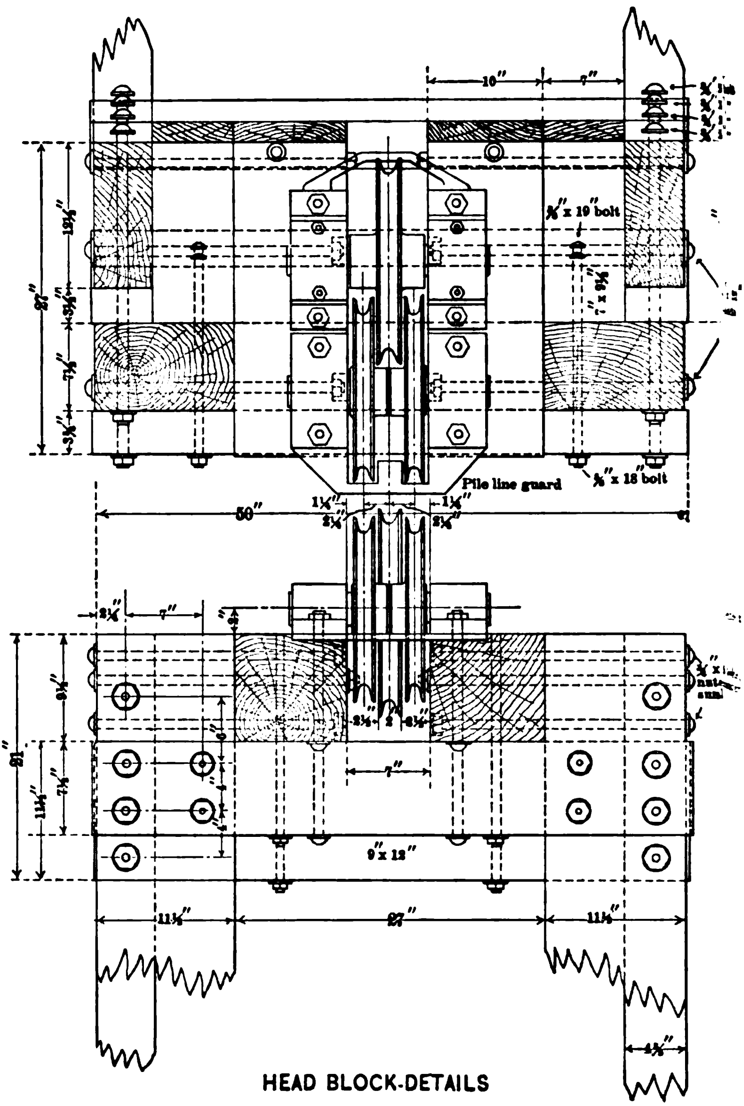
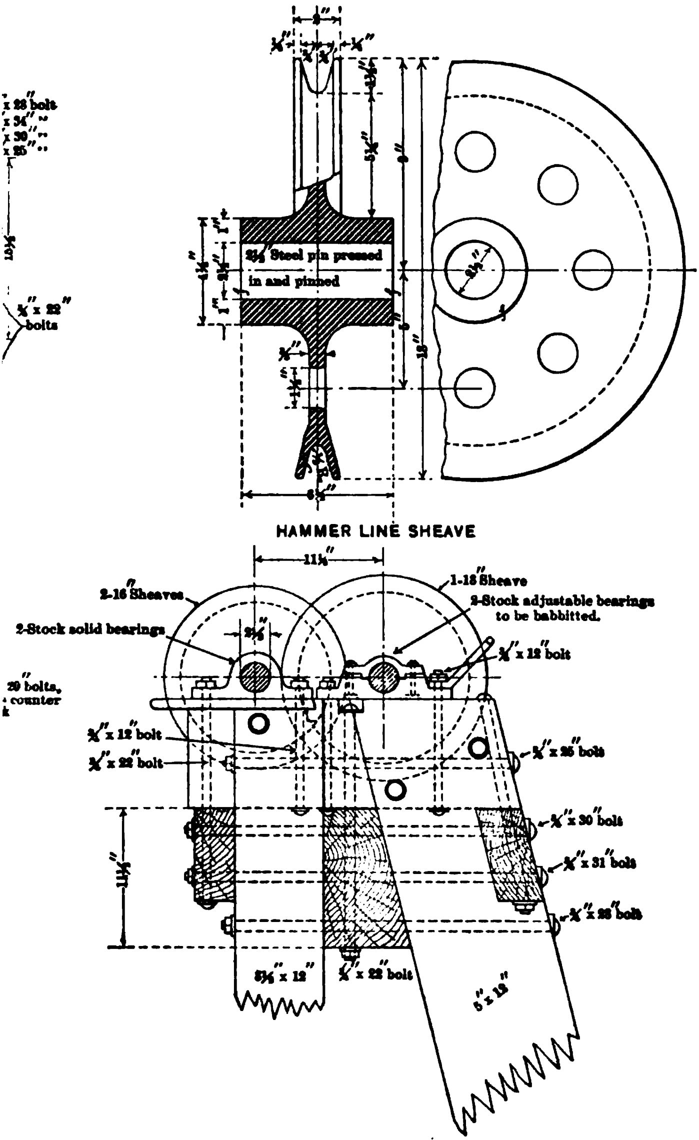


FIG. 48.—HEAD BLOC



x, U. S. Driver. *(To face page 70.)*

paint of pleasing color. The deck should be kept covered with 1-inch boards as a protection from the calks in the soles of the workmen's shoes.

The details of the jetting pipes, hose, and jets will be found in Chapter V on Jetting Piles.

The specifications for the government floating driver are given in full in Appendix VII.

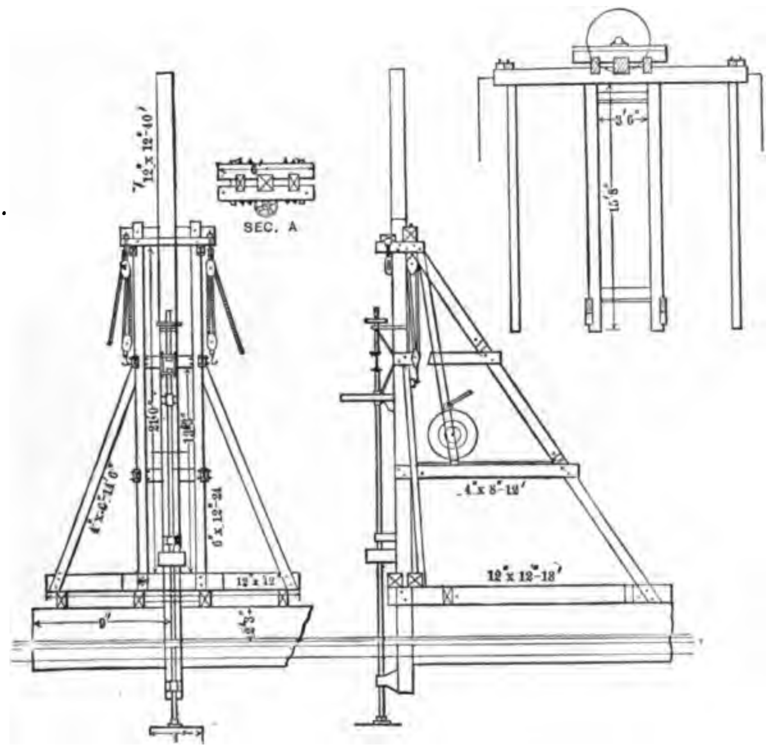


FIG. 49.—MACHINE FOR SAWING OFF PILES UNDER WATER.

Mention has been made of the use of a rock drill as a Nasmyth hammer, on the Great Kanawha River coffer-dams; and where any amount of driving is to be done it will certainly be wise to use a hammer of the Nasmyth type.

The guide-piles of a coffer-dam should always be driven with the idea of using them as a support for pumps, engines, derricks, and the like, although it will often be found cheaper to rig up on flatboats when there is danger from floods. In determining what load a pile will carry from this source, or when driven as a founda-

tion pile to support the masonry, Wellington's formula is at once the most accurate and the easiest to remember and use. For a drop-hammer, multiply twice the weight of the hammer in pounds by the drop in feet and divide by the last sinking in inches plus one, and the result is the load in pounds the pile will carry, with a factor of six for safety. This is easily remembered as $2wh$ over $s+1$, and is always ready for use. For the steam-hammer the form is $2wh$ over $s+0.1$, the "wh" representing the dynamic effect of the hammer.

Where piles have been firmly driven and they are to be removed when the work is done they can be cut off under the water by a machine similar to Fig. 49, which can be operated from a barge. The description in the *Engineering News* gives but little information in addition to the drawing. The shaft works in cast-iron sleeves

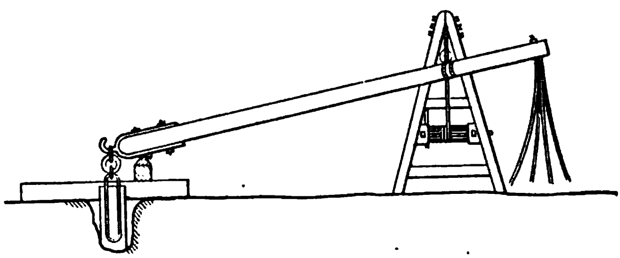


FIG. 50.—PILE-PULLING LEVER. AFTER CRESY.

attached to a timber, which slides in the leads, this being operated by the winch shown in side elevation. The final adjustment is made by the hand-wheel on the 3-foot adjusting screw. Where the piles are not so solidly driven they can be pulled out with a lever, an old form of which is given by Cresy (Fig. 50). In place of the pin and links, a chain closely wrapped around the top of the pile is usually made use of.

The apparatus used on the New York State canal work (Fig. 51) consisted of a strong frame mounted on a scow, from which was suspended a heavy set of falls to attach to the chain wrapped around the head of the pile. The pulling was done by an engine placed on the scow.

The construction of coffer-dams with sheet-piling has led to the use of a number of forms of sheet-piles, some of which are driven only as a protection to the puddle, while others are nearly or quite water-tight in themselves. The principal forms are shown in Fig. 52, the simplest form being plank of some considerable thickness

(*a*), for which Stevenson specified $4\frac{1}{2}$ inches by not exceeding 9 inches in width for the Hutcheson Bridge. The points are sharpened as at (*i*) so they will draw together in driving, and as at (*j*), to cause them to drive straight and easy. The same principle is embodied in the patent metal point shown at (*k*), which is used to protect the point when driving through coarse gravel.

The piles at Buda Pesth were increased to 15 inches square in order to resist the pressure brought upon the sides of the dam by the puddle, the water, and also by the ice. Flat plank are also used by driving two or more rows as at (*b*), the second and third

FIG 51.—PILE-PULLING SCOW, NEW YORK STATE CANALS.

rows being used to close the cracks in the main row of piles and retain the puddle. An example of this will be given in the next article, where it was used on the Michigan Central Railway. The extra rows may be of thinner plank if they can be driven.

Mention has already been made, incidentally, of the use of V-shaped tongue-and-groove piling (*c*), on the Union Pacific Railway. This may be made on a beveled saw-table, the saw cutting half through the plank from opposite sides at each cut. This will produce a reasonably tight wall, if care is used in driving and if the points are sharpened to draw them together and make tight joints.

Ordinary tongue-and groove piling (*d*) is frequently used, but a more frequent form is that shown at (*e*), like that used on the Robinson circular dam. The two pieces forming the groove and

the piece for the tongue are spiked to the 9×12 and 6-inch spikes sloping upward. A sheet-pile dam on another pier of the Arthur Kill Bridge employed piling in which the grooves were made by making two saw cuts and cleaning out between with a chisel, the tongue being formed in the same manner as at (*f*), the tongue being spiked in one side.

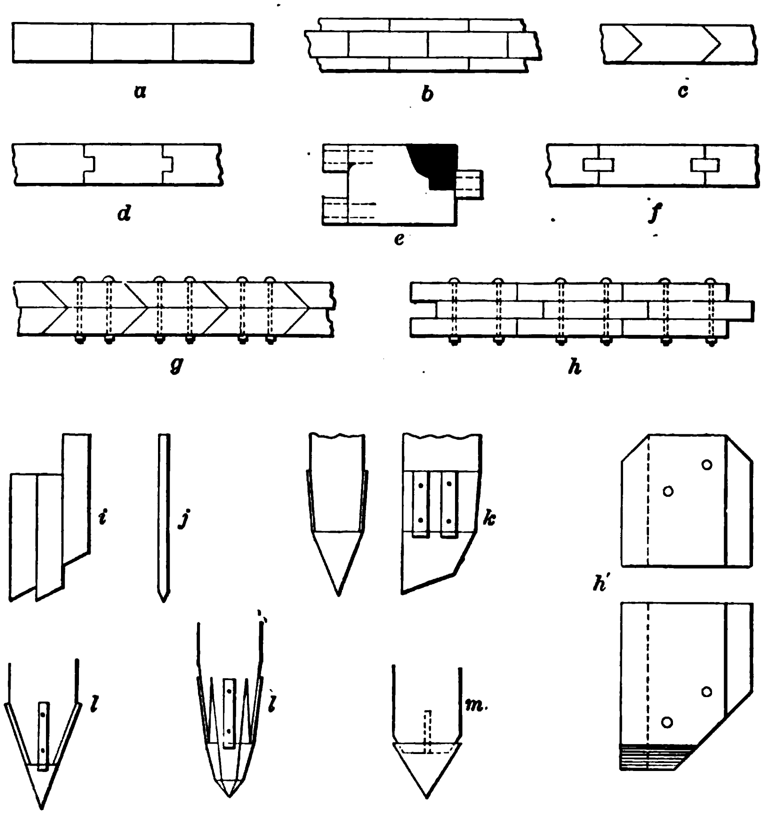


FIG. 52.—SHEET-PILES AND SHEET-PILE DETAILS.

A method which is not often employed is shown at (*f*), two grooves being made in the sheet-pile and a key driven after the piles are down. Should the piles not drive in perfect line, and the groove fail to match, the method will not be found to be a success.

Sheet-piling formed of two or more planks bolted together is being extensively used, one of them (*g*) being formed by two planks sawed with beveled edges and bolted together to form a pile similar to (*c*). This forms a pile which will drive easily on account of having

some size and which will require fewer supports in the shape of waling-pieces.

TABLE III.—PILES AS COLUMNS. SAFE LOAD PER SQUARE FOOT OF SECTIONAL AREA.

Authority.	Unit Values. Lbs. per Square Inch.		Safe Load per Sq. Ft. Lbs.
Rankine and Mahan	1000		144,000
Peronnet	786 to 990		113,184 to 142,560
Stoney	$\frac{1}{10}$ crushing wt. of dry timber.		
	Elm	1000	144,000
	Ash	860	123,840
	Beech	800	115,200
(See page 109)	Spruce	650	93,600
	Cedar	610	87,840
	Oak	600	86,400
	Yellow Pine	538	77,472

Several examples already given describe the use of Wakefield patent sheet-piling (*h*), the method of sharpening being shown at (*h'*). This is constructed of three layers of plank from 1 to 4 inches thick, according to the pressure to be sustained. The center plank must be sized to keep the tongue and groove uniform, and the plank are bolted together with six bolts for a length of from 16 to 20 feet, two bolts near each end and two intermediate. For long piles, spikes should be driven between the bolts. The bolts vary from $\frac{3}{8}$ inch for 1-inch plank to $\frac{3}{4}$ inch for 4-inch plank. A coffer-dam constructed with this piling is shown in process of construction in Fig. 53, for the foundations of Charlestown Bridge near Boston. A description of this will be given in the next chapter.

Pile-shoes for use on round or square piles are shown at (*l*) and (*m*), (*l*) being patent forms. Straps of bar iron are used in many cases with success, for main piles, and sheet iron of $\frac{1}{8}$ inch thickness, bent to a " V " and spiked on, is often all that is necessary when shoes must be used on sheet-piles.

The thickness of sheet-piling should be sufficient to prevent the plank from bulging and should be calculated to stand a water pressure due to the depth, and for a span equal to the distance between the waling-timbers or other supports. This would necessitate wales every 6 feet for 3-inch plank under 5-feet head, or wales every 3 feet for a 21-feet head. Plank $4\frac{1}{2}$ inches thick would require wales every 7 feet under a 9-feet head, or every 5 feet for an 18-feet head.

Timbers 9 inches thick will carry 9 feet under a 20-feet head, while the 15-inch timbers of the Buda Pesth dam would carry 12 feet under a 33-feet head.

FIG. 53.—CHARLESTOWN BRIDGE. DRIVING WAKEFIELD SHEET-PILING.

Good timber should always be employed if it can be procured, or, if faulty stuff must be used, allowance must be made by using thicker piles and by placing the wales closer together.

The timber used for piling in various parts of the United States is Douglas fir, long-leaf pine, tamarack, redwood, cypress, and the best grades of cedar and oak. Redwood and cedar do not rot so quickly as other woods, but lack in strength for structures and for

hard driving. For temporary structures hemlock, maple, elm, and other less reliable woods, are used. The usual specification requires that piles be cut from sound, live trees, and must be free from the usual timber defects, except that a small amount of heart rot be allowed in cedar.

TABLE IV.—SAFE LOAD IN TONS ON PILES $= \frac{2wh}{s+1}$.

Weight Hammer.	Last Sinking in Inches under 15 Ft. Drop.									
	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
1200	15.0	12.9	11.3	10.0	9.0	8.2	7.5	6.9	6.4	6.0
1600	20.0	17.2	15.0	13.4	12.0	10.9	10.0	9.3	8.6	8.0
2000	25.0	21.4	18.8	16.7	15.0	13.6	12.5	11.6	10.7	10.0
2400	30.0	25.7	22.5	20.0	18.0	16.4	15.0	13.9	12.9	12.0
2800	35.0	30.0	26.3	23.4	21.0	19.1	17.5	16.2	15.0	14.0
3000	37.5	32.2	28.1	25.0	22.5	20.4	18.8	17.3	16.1	15.0
3200	40.0	34.3	30.0	26.7	24.0	21.8	20.0	18.5	17.2	16.0
3400	42.5	36.4	31.9	28.4	25.5	23.2	21.3	19.6	18.2	17.0
3600	45.0	38.5	33.8	30.0	27.0	24.6	22.5	20.8	19.3	18.0
3800	47.5	40.7	35.7	31.7	28.5	25.9	23.8	21.9	20.4	19.0
4000	50.0	42.8	37.5	33.4	30.0	27.3	25.0	23.1	21.5	20.0
4200	.....	45.0	39.4	35.0	31.5	28.6	26.3	24.2	22.5	21.0
4600	.....	49.3	43.1	38.4	34.5	31.4	28.8	26.6	24.6	23.0
5000	.....	.....	46.9	41.7	37.5	34.1	31.3	28.8	26.8	25.0

Never use loads greater than those above upper black line, except in emergency, when the lower black line may be used as the limit. In cases of rare emergency values below lower line may be used, if all conditions are certainly known.

Usually the piles are to be peeled, but often tight-bark winter-cut piles are required, as it is protection in some cases, like in salt water against teredo, where the bark will protect for a period of from one to two years, if there are no breaks or abrasions of the bark. In some particular cases the piles are required to be absolutely straight, but this is a very rigid demand and mostly they are specified to be so that a line stretched from end to end of a pile will not fall outside the stick.

The size of piles is usually specified to be not less than an 8-inch top or point and not less than a 14-inch butt. They should not run over 24-inch butts, or else they would have to be slabbed off to go in the pile-driver leads. Where piles from 70 to 120 feet

long are used the tops are often allowed as small as 7 inches, and in rare cases 5 inches or 6 inches diameter.

The driving of piles should never be carried to a point where there is danger of brooming or shattering the stick, and this must usually be left to the judgment of those in charge of the work. Fig. 54 shows the results of overdriving on the Vancouver, Wash., bridge. The best results will be obtained by using a hammer of from 3400 pounds to 4200 pounds, with a short drop. The hammer should be of a short pattern with the weight mostly concentrated

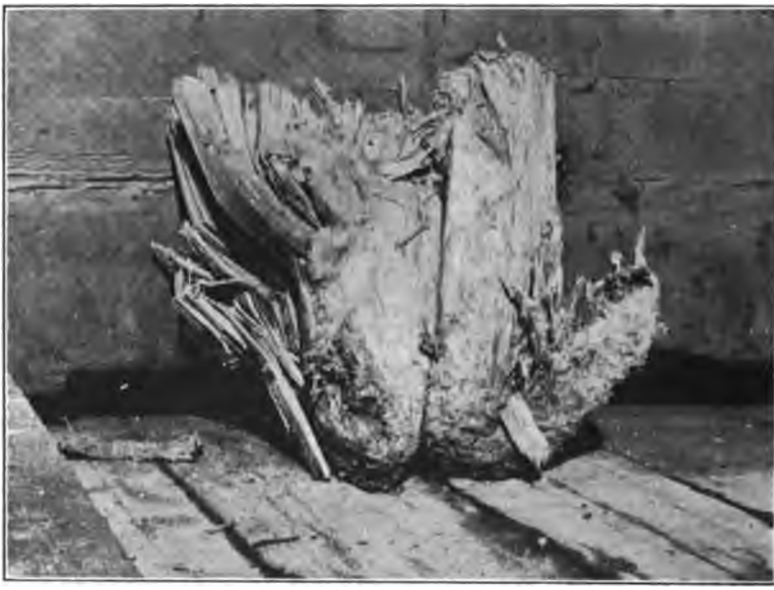


FIG. 54.—EFFECT OF OVERDRIVING PILES.

in the ball or bottom end. In sand, quicksand, soft clay or light gravel a steam-hammer striking from 60 to 130 blows per minute will be found more effective than a drop-hammer, and the weight should be from 5 tons upwards. The steam-hammer will usually make it possible to avoid the use of jets in this class of material, and as jetting nearly doubles the cost of pile-driving it should be avoided whenever some other method can be employed.

A recessed base on the steam-hammer acts as a cap on the head of the pile and avoids the need for pile rings, although the pile heads must be shaped up to fit the base. With the drop-hammer a follower cap, Fig. 55, may be used to avoid the use of pile rings. Where piles

are to be driven below water, the base casting on the follower, Fig. 56, can have the same shaped bonnet to fit the head of the piles, and when the piles have been followed down to the proper elevation ready for pouring the concrete around them, it will be unnecessary to cut them off under water, as the heads will be uninjured and in proper condition for carrying the load. The author has made experiments on this method above water and has had the heads of

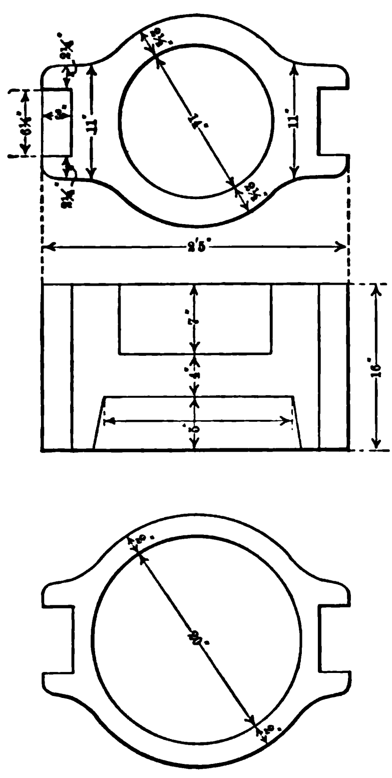


FIG. 55.—FOLLOWER CAP FOR WOOD PILES.

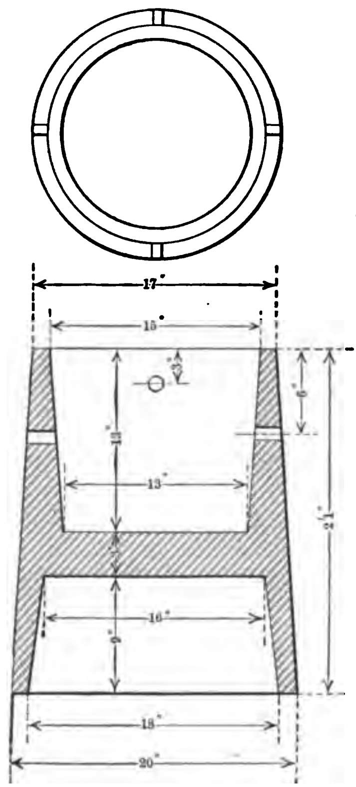


FIG. 56.—FOLLOWER BASE CASTING.

piles cut off under water by a diver after being driven by this method and found them in better condition than before, inasmuch as the fiber of the timber has been compressed and in better shape to take the bearing.

Pile rings about $\frac{7}{8} \times 2\frac{1}{2}$ inches must nearly always be used with the drop-hammer unless the driving is very soft or unless the bonnet cap is employed.

Pile shoes as shown in Fig. 52 (*l*) and (*m*) may be used in hard clay and compact gravel with good results, but in hardpan and cemented gravel there is every likelihood of the pile splitting out around the shoe and brooming up worse than if nothing was used. The triangular cast point (Fig. 57) used on some recent work in hard clay and cemented gravel worked much better, the sharp corners cutting into the material, so that a penetration of several feet was obtained. For ordinary compact material it is usually necessary only to point up the end of the pile as shown in Fig. 52, leaving off the point (*l*).

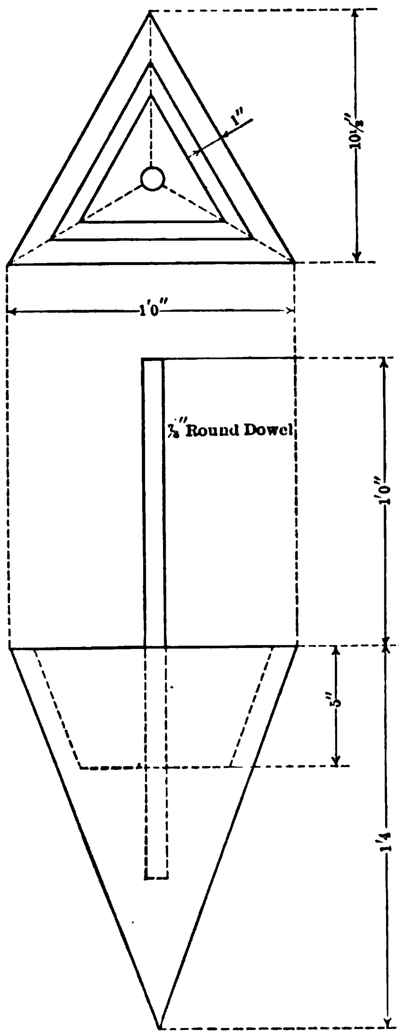


FIG. 57.—TRIANGULAR CAST PILE POINT.

Piles have been driven butt down in very soft material, to get better bearing and additional frictional area, but it is objectionable, as the small end of the pile will not stand hammering, it will spring in driving; the top is too small for proper bearing for the caps, and the part of the pile out of ground is the smallest diameter and will not carry the proper load in many cases as a timber column.

The use of swinging or pendulum leads is very often desirable where batter piles are to be driven in a trestle, and the rig shown in Fig. 58 is a very simple one to construct and very efficient. Where brace piles are to be driven under a wharf with a floating driver, the rig shown in Fig. 59 may be used, being hung on the face of the driver leads, and can be moved up and down to various elevations required to drive the braces, or to allow for rise and fall of the tide.

The ordinary crew for a land driver is six men including the foreman, and for a floating driver from seven to nine men, including the foreman and boom man. On some work an extra boom man is required and often extra men for such work as heading up the piles; for jetting work from two to four extra men are required to handle the jets. The cost of driving piles will be discussed in the chapters on Cost of Work at the end of this volume.

Pulling piles is done by using double or triple wire-rope blocks for falls on the face of the leads, the pile being gripped by a chain or

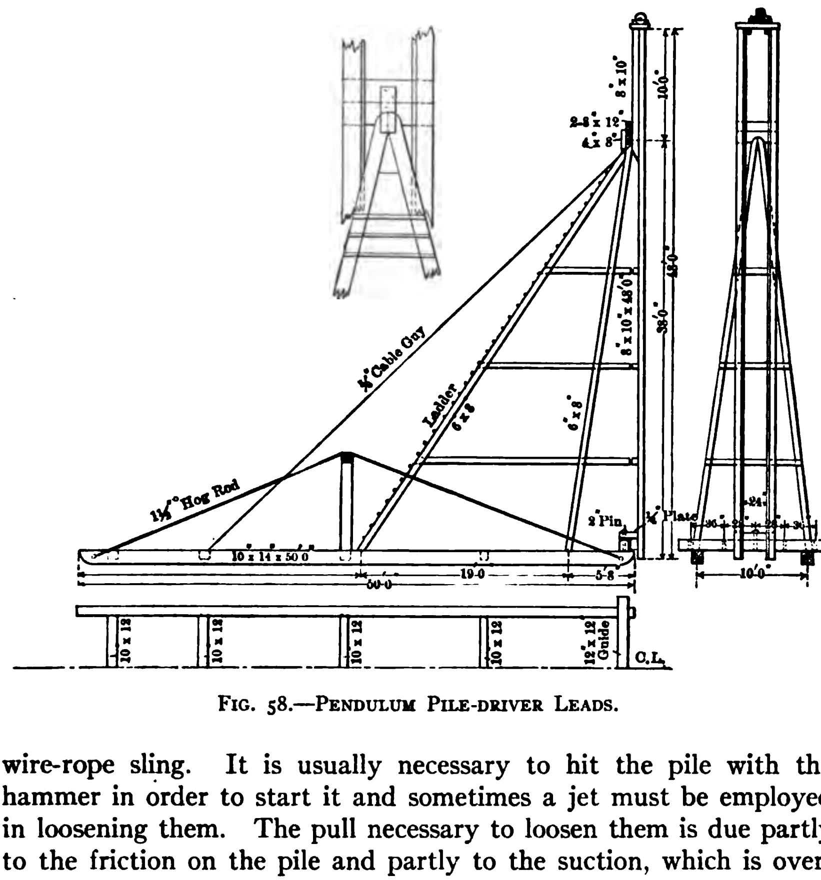


FIG. 58.—PENDULUM PILE-DRIVER LEADS.

wire-rope sling. It is usually necessary to hit the pile with the hammer in order to start it and sometimes a jet must be employed in loosening them. The pull necessary to loosen them is due partly to the friction on the pile and partly to the suction, which is overcome by taking a pull with the driver and holding on until the pile lets go. This friction and suction has been found in some cases to amount to as much as 1800 pounds per square foot on the surface of the pile, but where the friction is taken account of in the load the pile will carry, it should not be taken at over 500 pounds per square

foot in firm sand or clay and will not amount to over 120 pounds in silt.

Concrete piling have come into such general use that they can no longer be considered an experiment, and are used without much question wherever a permanent foundation is required.

They should always be used where there is any danger of wood piles decaying, that is, where piles will not constantly keep wet. Where some more permanent protection against limnora or teredo

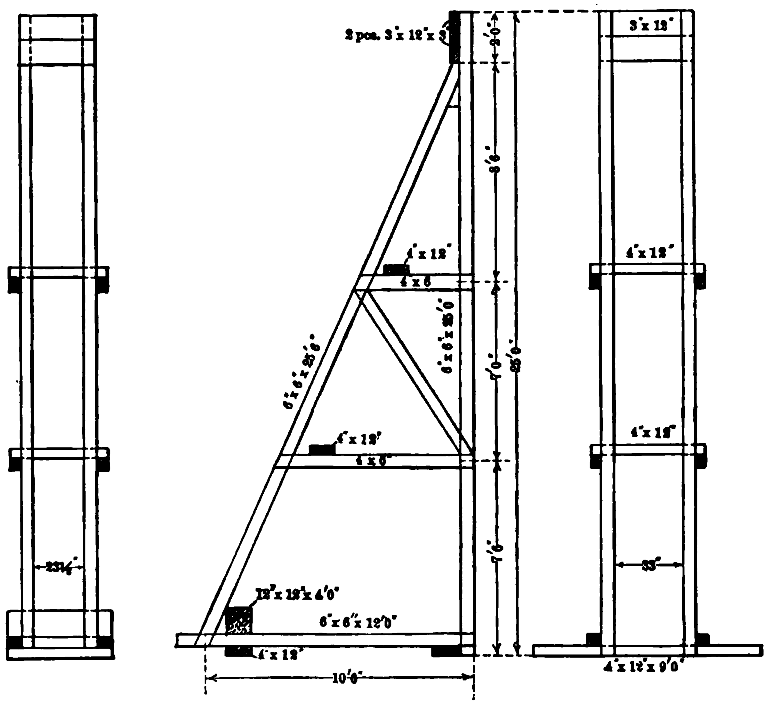


FIG. 59.—BATTER PILE-DRIVER LEADS.

than that offered by creosoting, is desired, concrete piles are very often the solution of the problem.

The Corrugated or Gilbreath pile is one of the best-known forms (Fig. 60), and the corrugations afford considerable additional frictional surface where piles are being driven into a soft bottom. This pile is a moulded pile, being usually poured into horizontal forms on the ground, the reinforcing being first placed in the form. The amount of reinforcing to be put into piles up to 35 or 40 feet is only enough for ordinary handling and driving. For the East Twenty-

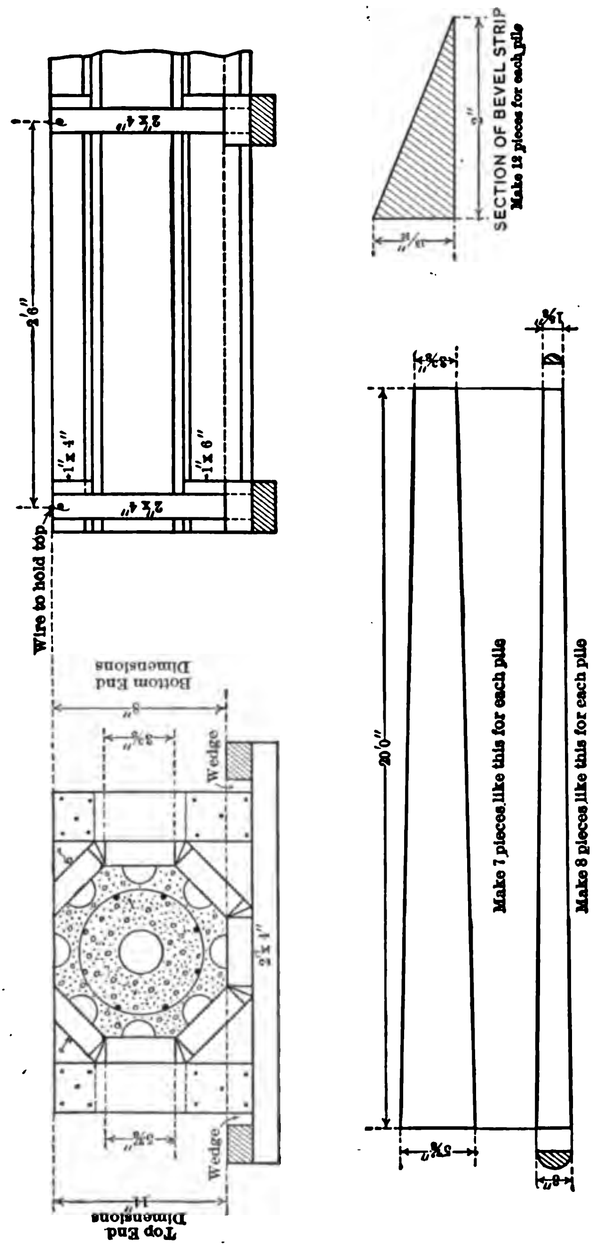


FIG. 60.—CORRUGATED CONCRETE PILE FORMS.

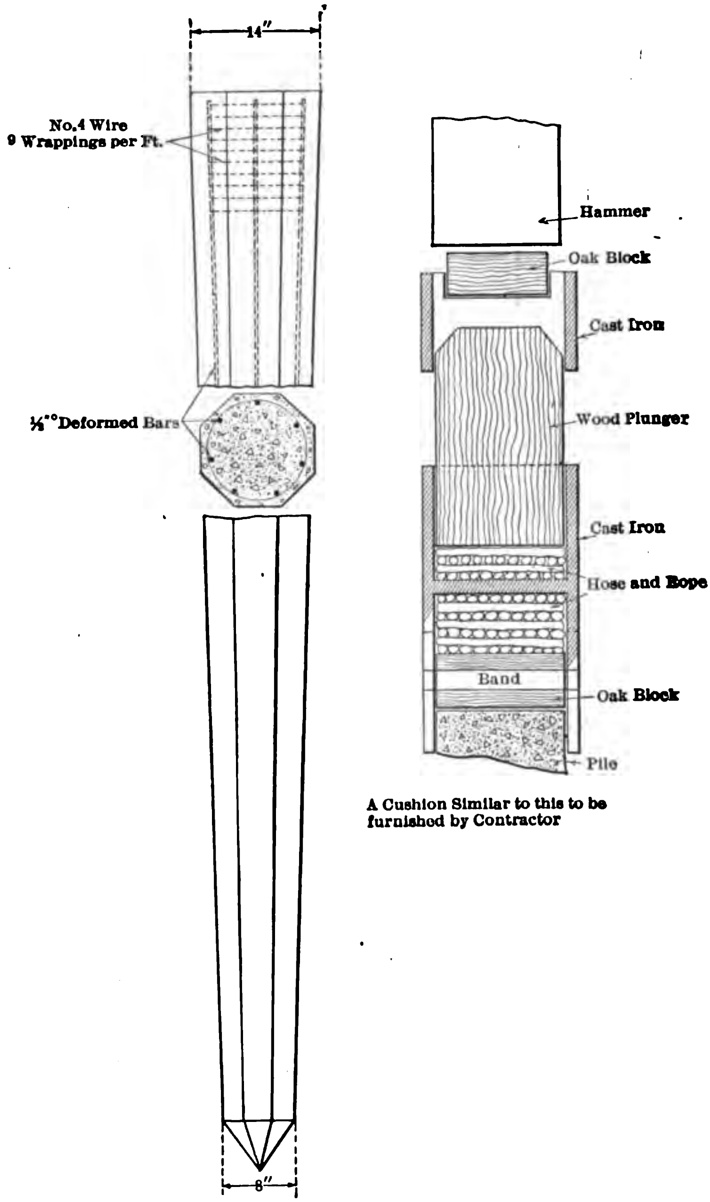


FIG. 61.—REINFORCED CONCRETE PILE AND DRIVING CAP.

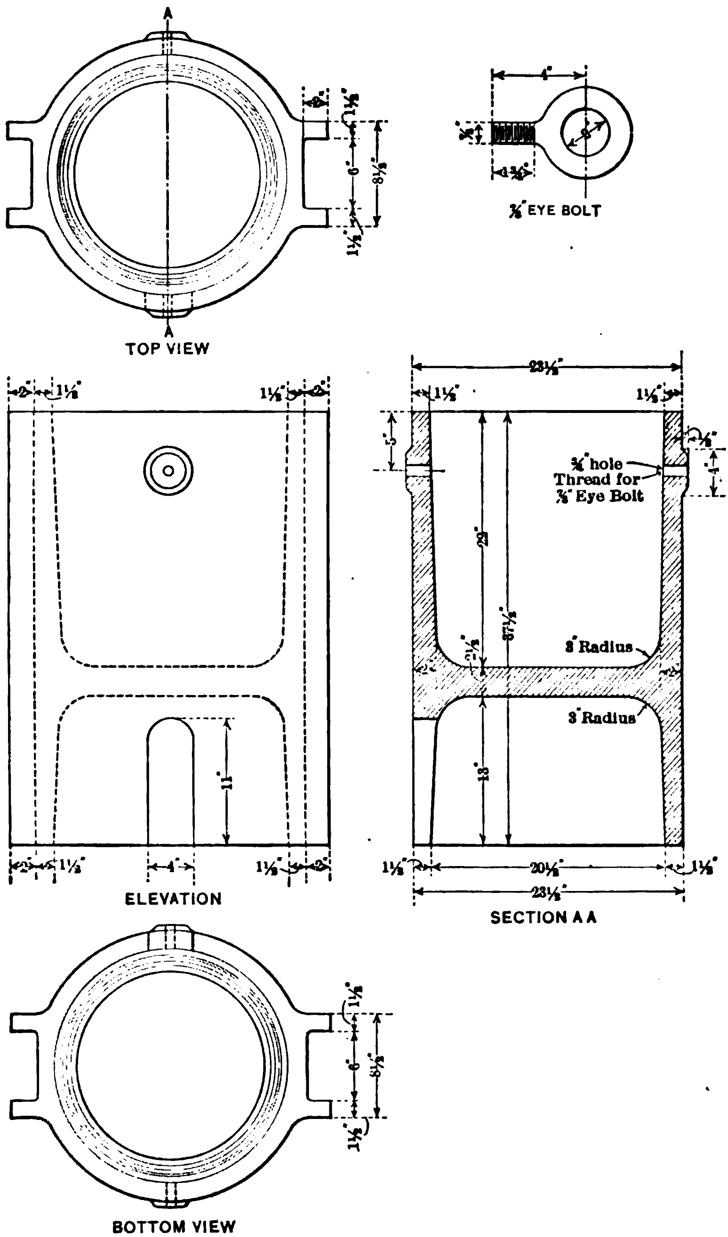


FIG. 62.—CASTING FOR CONCRETE PILE CAP.

first Street Viaduct in Portland, Oregon, the author used piles with seven half-inch round deformed bars lengthwise and nine wrappings per vertical foot of No. 4 wire (Fig. 61). The driving head specified for this is shown in Fig. 61 and is substantially that specified by the patentees of the Corrugated pile, the details of the steel casting for the cap made for this is shown in Fig. 62. The complete specifications for these piles required them to be cast in a horizontal position on the ground, of 1 : 2 : 4 concrete, and driven with a 3000-pound steam drop-hammer, and a three-foot drop. "The pile shall be allowed to set for thirty days before driving. Safe load assumed for each pile to be 22 tons, and the contractor will be required to

FIG. 63.—CONCRETE PILE CURING YARDS.

test four piles; each test pile shall be loaded with 22 tons for 48 hours and shall show no settlement. Each pile shall then be loaded with 35 tons and shall not show more than ¼-inch settlement after the load has been applied for 48 hours. Should the test piles fail to sustain the load as specified, the contractor must increase the length of the piling, so that in the opinion of the engineer the piling used will safely carry a load equivalent to 22 tons for each pile shown on the plans."

The piling used were as shown, but corrugations were added and the jet pipe cast in the center of each pile. (Fig. 60). To avoid delay, while the piling were curing in the yards (Fig. 63), test-piles of timber were driven and the ordinary formula applied to determine

their bearing capacity. The material was a sandy, slate-colored clay, moderately soft, and the test-piles were found to give satisfactory results, without the necessity of using longer ones than 30 feet. With frequent wetting down, the piles cured enough for handling

FIG. 64.—TWENTY-FIRST STREET VIADUCT, PORTLAND, ORE.

without damage in about 20 days. They were driven by a 3000-pound drop-hammer, and the jet operated from a $7 \times 4\frac{1}{2} \times 10$ duplex pump, giving a pressure at the pump of about 175 pounds.

The actual driving developed the fact that the piles were not as easily damaged as wood piles, and also that for the material to be

penetrated, the center jet was not so good nor so effective as a separate jet-pipe outside the piles, and this was used for practically the entire work. The piling were made and driven under the direction of H. W. Holmes, M. Am. Soc. C. E., engineer in charge of the entire structure (Fig. 64), and also the designer.

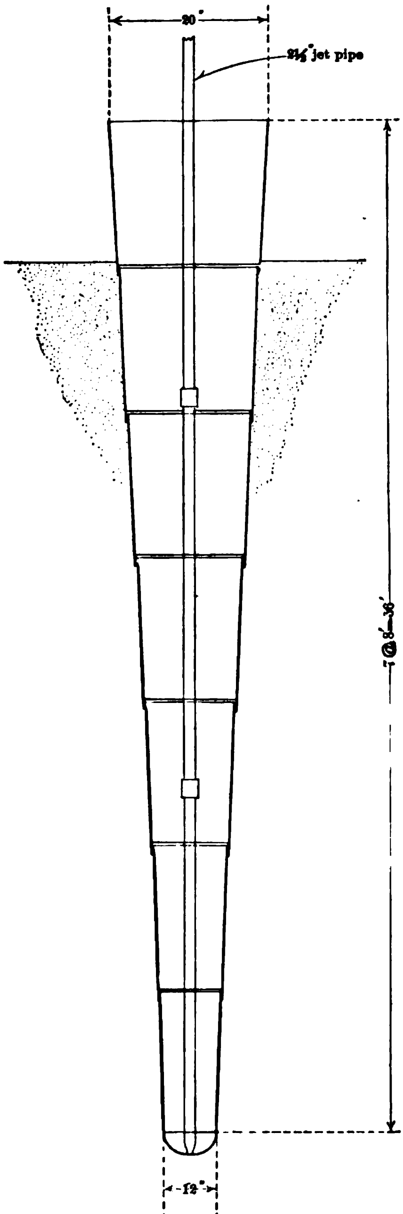


FIG. 65.—RAYMOND CONCRETE PILES.

The usual reinforcing for the corrugated pile consists of Clinton wire-cloth placed just inside the corrugations, the cloth having a mesh of 3×12 inches, the 12-inch mesh vertical, with No. 3 wire lengthwise of the piles and No. 10 wire around the inside. This reinforcement is only sufficient, as has been stated, for ordinary lengths and handling; for rough usage and for resisting bending stresses it must be increased.

There are many other forms of piles cast before driving, but the principle is the same in all of them. The piles that are built in place are of three different types, the Raymond, the Simplex, and the Clark. Using these, care must be taken not to drive piles too close to those already driven, until they have time to set, as in many instances they will be practically sheared off when the concrete is green. This may be avoided by skipping rows and then filling in later on when the concrete is fully set.

The Raymond pile (Fig. 65) has a steel shell of conical shape

into which a conical steel core is placed for driving. After it is driven, the core is withdrawn and the steel shell, which is heavy enough to keep its shape, is filled with concrete. If reinforcing is used, it is placed in the shell before the concrete is poured. This usually consists of a center rod about $1\frac{1}{2}$ inch round, and three $\frac{3}{4}$-inch rods near the circumference. The use of more reinforcing would assist in keeping the pile intact when driving adjacent piles.

The Simplex pile has a wrought-iron or steel-pipe form which is driven, being heavy enough to stand severe driving and it is fitted with a concrete or steel point and a hardwood driving-head. After the pipe is driven to the required depth it is filled with concrete.

The Clark pile is similar, only it has a jointed shell of $\frac{3}{8}$ inch thickness or heavier; the joint is made by an inside coupling. The pipe is driven by steam-hammer and jet-pipe. After rock or the required depth is reached, the pipe is washed out with the jet and the pipe filled with neat cement grout. Reinforcing may be placed as required.

Where sheet-piling is to be retained as part of the permanent structure, it may be made of rectangular moulded concrete piling,

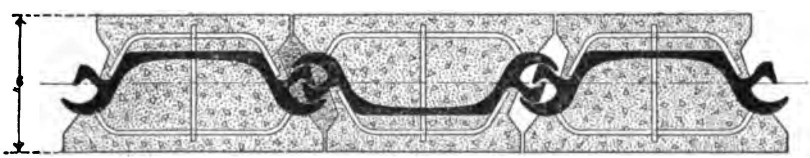


FIG. 66.—LACKAWANNA REINFORCED CONCRETE SHEET PILING.

with reinforcing as may be necessary to sustain the load on the piling, either vertical or transverse.

The use of metal sheet-piling for this purpose, protected by concrete, is a very satisfactory solution in many cases. The Lackawanna sheet-piling used for this purpose are shown in Figs. 66, 67, and 68, the forms for moulding these piles being shown in Fig. 67, while the section of the piles is shown in Fig. 66. It is important that all surfaces of forms in contact with concrete must be smooth; preferably machine-planed. All other surfaces need be planed only enough to insure good joints. All outside surfaces may be left undressed. Stock size lumber should be used wherever possible. Forms can be extended to fit any length of piling by lengthening the top, bottom and side planks and the running strips; forms can be shortened by placing the bottom end blocks further in the form. The use of this piling in constructing a 21-foot levee is shown in Fig. 68.

The use of ordinary reinforced rectangular concrete sheet-piling is described in Chapter XXVII, where it was used on the piers in

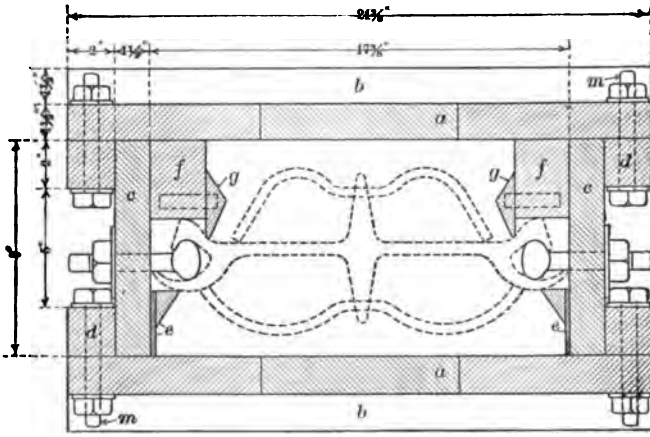


FIG. 67.—LACKAWANNA SHEET PILING FORMS.

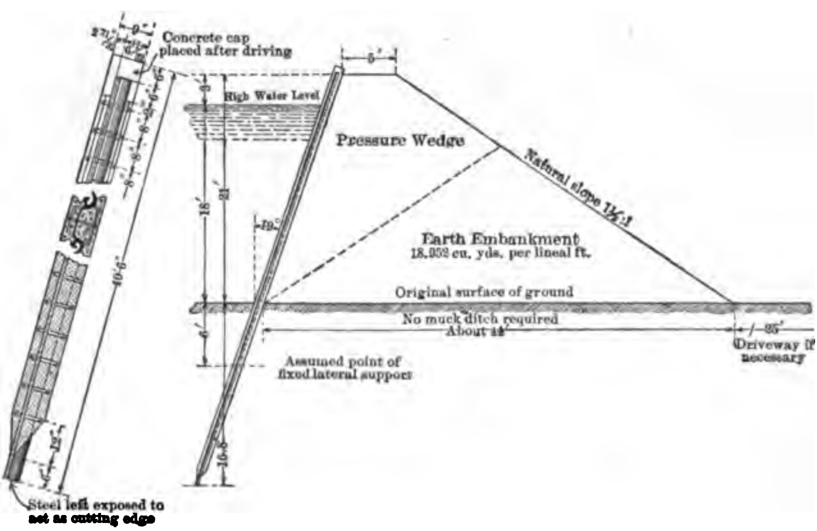


FIG. 68.—LACKAWANNA SHEET PILE LEVEE WALL.

the improvement of Baltimore harbor. The cost of this class of work will be very similar to that for ordinary reinforced concrete piles, as given in Chapter XXXII, on the Cost of Foundation Work.

CHAPTER V

JETTING PILES

THE use of water jets in the sinking or driving of piles has been referred to in several places in this work, but it is the purpose of this chapter to give in proper detail the methods and plant necessary for the best results in work of this character. The first use of water jets in pile-driving was probably in 1852, at Matagorda Bay, Texas, following the idea of Gen., then Lieut., Geo. B. McClellan, Corps of Engineers, U. S. A. The nozzle was placed close to the point of the piles, being connected by an ordinary hose to a hand force pump. The method was very crude, but doubtless of much service in light material.

Hollow cast-iron piles with a disc base were sunk by a water jet from a hand pump in Chesapeake Bay in 1854, for the foundation of the Pungateague Light.

The ordinary jetting carried on for many years was only as an aid to driving with a drop-hammer and the water was usually supplied by a small duplex steam pump, about $6 \times 4 \times 6$ in size, supplying one jet through a 2-inch pipe. This is similar to the Sandy Lake jetting described in Chapter VII, where the supply hose was only $1\frac{1}{2}$ inch, the jet pipe $\frac{1}{2}$ inch, attached to the sheet piles, and with a $\frac{3}{8}$-inch nozzle.

The question of whether or not to attach the jet to the piles or to leave it loose is one about which engineers seem to differ widely, but in the author's opinion there can be but one result in the actual use, that is to leave the jet or jets loose.

With only one jet in use and that fastened to the pile with the nozzle at the point of the pile and to one side, there can be but one result—the pile will run or drive out of line. Where only one jet is used it is necessary to keep hammering the pile with short drops of the hammer, but before starting the pile, the jet should be run down into the bottom in the exact location where the pile is to be driven and to the full depth; then upon dropping the pile into the hole thus formed, it will go down under its own weight and the weight

of the hammer resting upon it, for a considerable distance or to full depth, or may then be driven in most cases to the required depth to get below scour and to carry the load, without further jetting. In case, however, it is necessary to have greater penetration than this will give, the jet can be run down the pile, first on one side and then on the other, to keep the pile going straight and to keep it properly lubricated with water its full length.

The size pump required for ordinary work may be as small as a $6 \times 4 \times 6$ or preferably a $7\frac{1}{2} \times 4\frac{1}{2} \times 10$, giving a pressure at the pump of from 150 pounds to 175 pounds per square inch, and supplying 140 gallons per minute through a 3-inch pipe. All pumps used in pumping salt water must be brass fitted and have brass-lined water cylinders. The supply is usually cut down somewhat by using $2\frac{1}{2}$-inch connecting and jet pipe, which should be double strength and have a nozzle made from a piece of pipe and drawn down to an orifice of from $\frac{7}{8}$ to $1\frac{1}{4}$ inch, the smaller-sized opening giving a greater cutting pressure, so that it is rarely advisable to use a nozzle larger than 1 inch diameter. This nozzle is shown in (*a*) **Fig.** 69, and one that works almost as good is shown at (*b*), made up of a pipe reducer and short nipple. The rose jet (*c*) and (*d*) has small holes bored around the sides to spread the water. This of course allows a greater discharge and the pump must be larger accordingly, if the main nozzle opening is kept the same size. Only a trial of this and other changes will determine which is best for any given material.

The pump shown on the floating driver (Fig. 44) is a $10 \times 6 \times 10$ center-packed plunger type, giving a supply of 220 gallons per minute at 200 pounds pressure through a 4-inch main pipe, and will give good results on two $2\frac{1}{2}$ inch jets in light material, with 1-inch nozzles, and is as large as is necessary for one jet in very compact material.

Many specifications require the use of two jets, or even three, all going at the same time in order to keep the pile going straight, and at the same time obtain the desired penetration. This will require a very much larger plant, and nothing less than a $12 \times 7 \times 10$ center-packed plunger pump will give satisfactory results for two jets operating in compact material, sand or light gravel, to considerable depths. This pump will deliver 300 gallons of water per minute through a 6-inch main pipe, at a pressure of from 175 pounds to 200 pounds per square inch. However, this kind of a plant for using two jets will add very greatly to the cost of the driving, practically doubling the cost of operating the driving plant. The pump scow shown in plan in Fig. 70 will have two boilers of locomotive type and 80 H.P. each, with fresh-water tank on salt-water work, feed pump,

injectors, feed-water heater, the jetting pump or pumps if a spare one is provided, together with the necessary pipes and valves. The

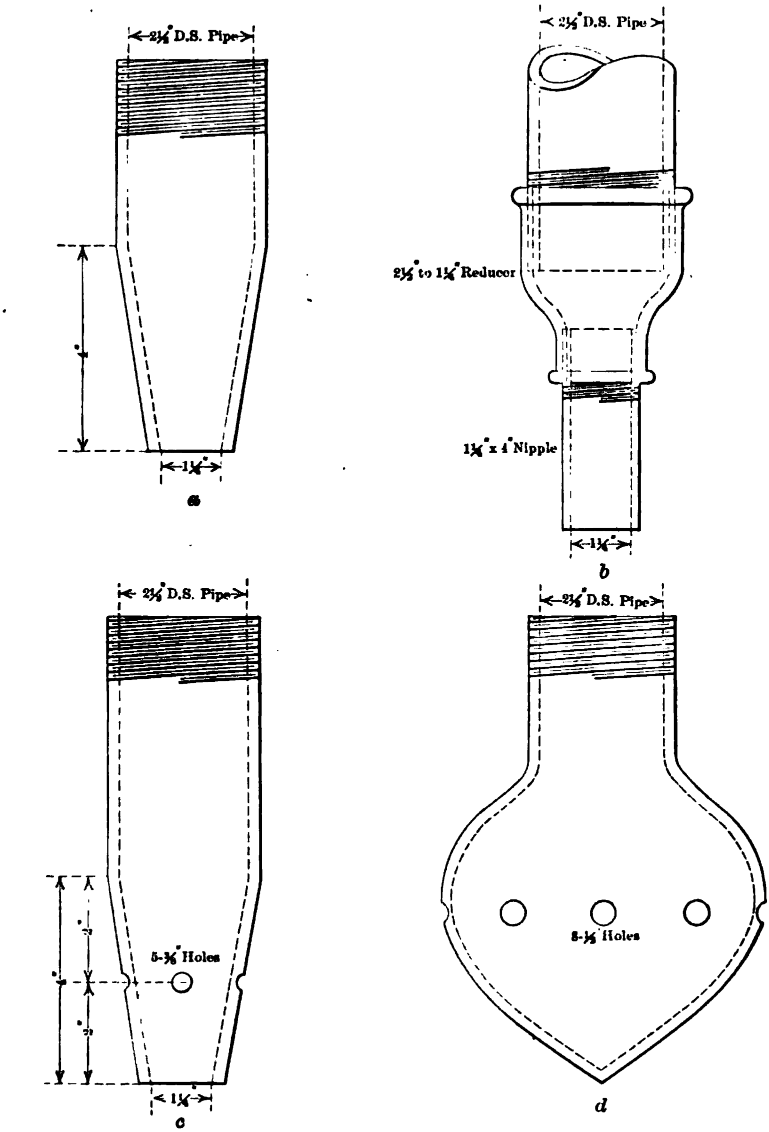


FIG. 69.—JETTING NOZZLES.

whole should be housed in by a light cover with at least 3 feet margin around the outside for walkway and handling lines. This plant,

moored alongside the driving plant, will require an extra engineer and fireman, and the driver will require two jet men to handle the jets and two nigger-head men to handle the jet lines. These extra

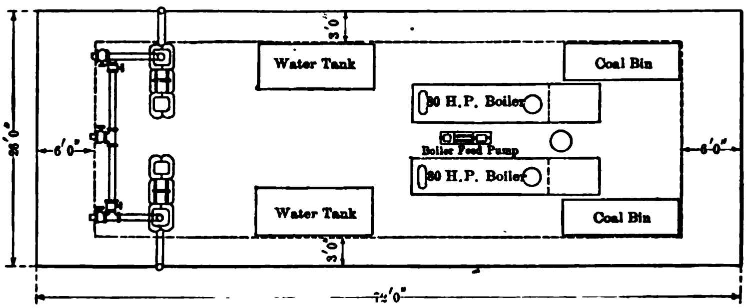


FIG. 70.—POWER SCOW FOR PUMPING.

men, with possibly a deck hand on the pump scow, together with fuel, supplies, and repairs will as stated above just about double the cost per day of operating the plant, and owing to the greater amount of plant to handle, the number of piles that can be driven in a given

FIG. 71.—DUPLEX PISTON PUMP.

time will be greatly reduced, so that jets should only be employed where absolutely necessary.

The sizes and capacities of locomotive and internal fired boilers Figs. 74, 75, and 76 are given in Tables VIII, IX, and X, the horse-power being given on the basis of 12 square feet of heating surface per horse-power, but usually figured on the basis of 10 square feet; the sizes and capacities of pumps (Figs. 71, 72 and 73) in Tables V,

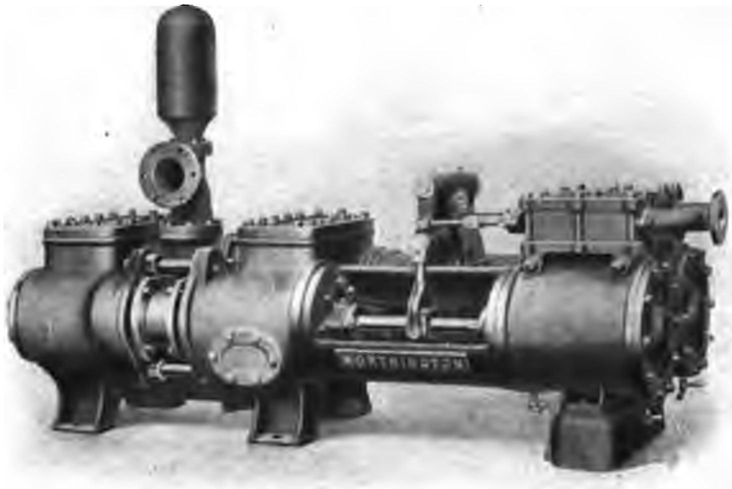


Fig. 72.—Duplex Center Packed Pump.

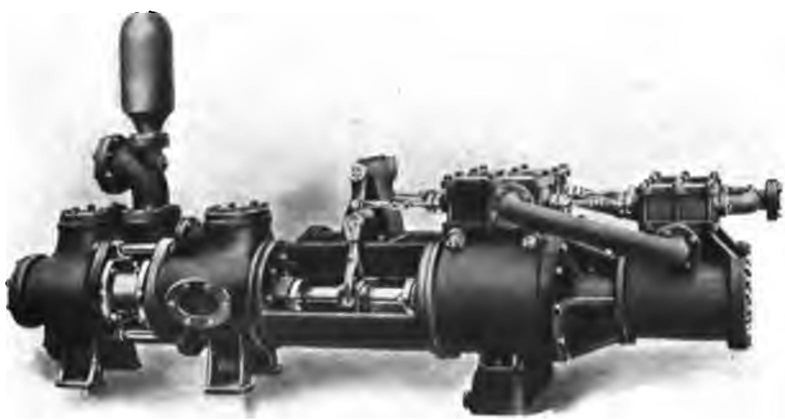


Fig. 73.—Compound Duplex Center Packed Pump.

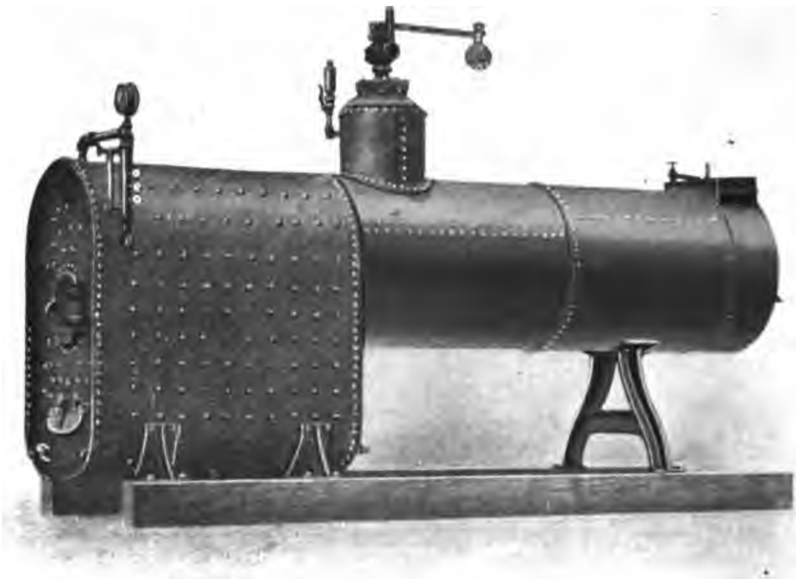


FIG. 74.—LOCOMOTIVE BOILER. WATER BOTTOM.

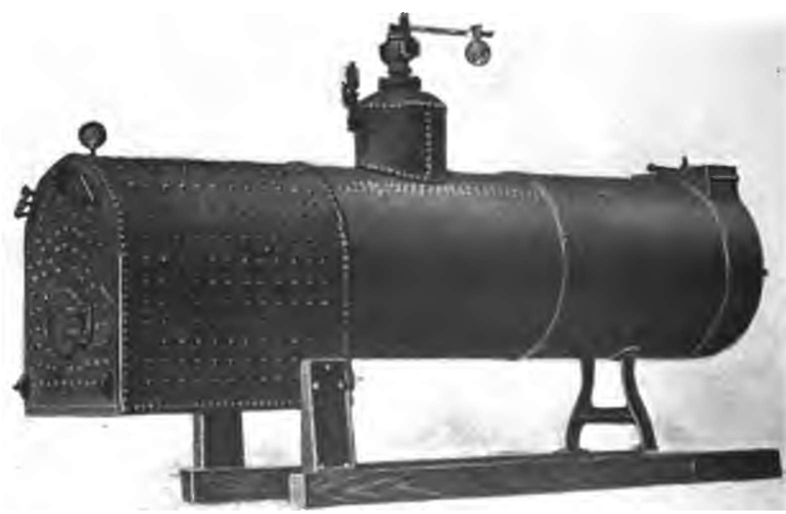


FIG. 75.—LOCOMOTIVE BOILER. OPEN BOTTOM.

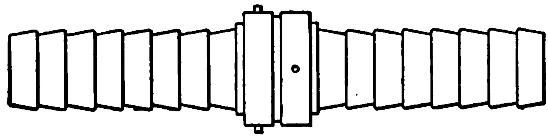
VI and VII, together with the boiler recommended by pump manufacturers for each size pump, increased 50 per cent, as the ordinary duplex pump is a "steam eater," there being an almost steady stream of steam running through it.

Where very compact sand, compact gravel, or clay is to be jetted, the center-packed plunger type of pump, with compound-steam

FIG. 76.—INTERNAL FIRED BOILER.

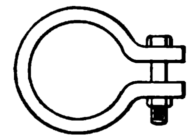
cylinders (Fig. 73) as listed in Table VIII, should be used, and boilers that will carry a pressure of from 150 to 175 pounds steam pressure. They will save about one-third the coal consumed by the piston-pumps. Such pumps will give an initial water pressure of 200 pounds per square inch, and will cut the harder material to better advantage than the ordinary duplex pump. They will require much higher quality of hose and much stronger couplings than for the pumps with lighter pressure. The couplings which are sold on stock hose

are of no value in jetting work, as they are only pressed in, and invariably blow out, so it is best to get special couplings with long



HOSE COUPLING

Fig. 77.



HOSE CLAMP

Fig. 78.

nipples to slip into the hose (Fig. 77) that will allow the use of two outside bands on each end (Fig. 78), for the ordinary duplex pump,

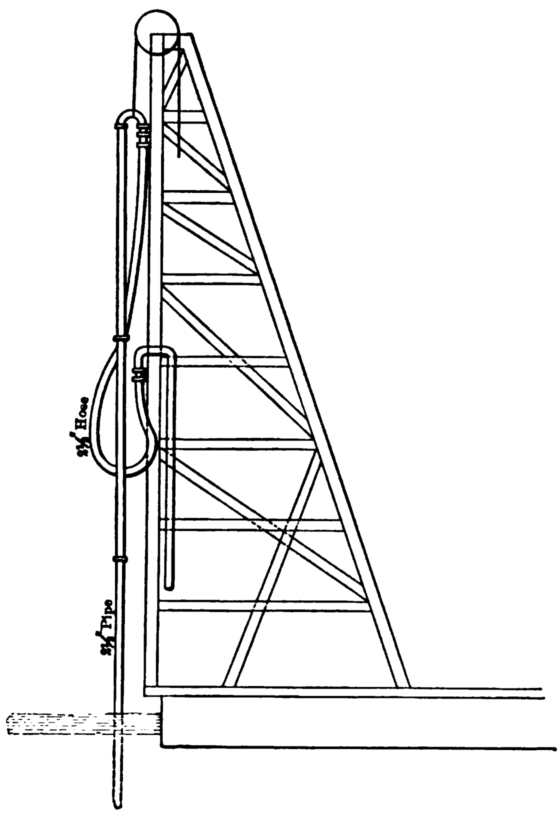


Fig. 79.—Jetting Pipe Details.

and three bands for the higher pressure pumps. These bands are drawn up with bolts as shown and will not cause trouble by blowing

out during the work. The couplings must be provided with pipe-threads. Unless the hose is of the wrapped type, it must be wrapped with burlap or gunny sack wherever it touches a timber or rubs on anything, as the vibrations will soon cut a hole.

The hose should be attached to the jet-pipe by a U and short nipple (Fig. 79) to avoid breaking the hose when under pressure. Where the jet is connected to the piping on a floating driver, large pipe the full size of the pump discharge should be carried up the leads one-third or one-half the height and the hose connected to nipples pointing downward (Fig. 79), so that the hose hangs in one big loop

TABLE V.—PISTON PUMP FOR 150 POUNDS WATER PRESSURE.

Diameter of Steam Cylinders.	Diameter of Water Pistons.	Length of Stroke.	Gallons per Revolution.	Max. Revolutions per Minute.	Max. Gallons per Minute.	Sizes of Pipes for Short Lengths to be Increased as Length Increases.				Approximate. Space Occupied. Feet and Inches.				Size Boilers. Required. for very Heavy. Work. H.P.
						Steam Pipe.	Exhaust Pipe.	Suction Pipe.	Delivery Pipe.	Length.		Width.		
2	1¼	2¾	.056	80	4.4	⅜	½	1	¾	1	9½	0	7	4
3	2	3	.155	80	12.4	⅜	½	1¼	1	2	0	0	9	5
3½	2¼	4	.275	75	23	⅜	¾	1½	1¼	1	9½	0	9	6
4½	2¾	4	.39	75	29	½	¾	2	1½	2	9	1	1	10
5¼	3½	5	.81	70	56	¾	1¼	2½	1½	3	5	1	4	15
6	4	6	1.25	65	81	1	1¼	3	2	3	7	1	5	20
7½	5	6	1.96	65	127	1½	2	4	3	3	9	1	10	35
7½	4½	10	2.60	54	141	1½	2	4	3	4	11	1	10	35
9	5¼	10	3.55	54	192	2	2½	4	3	5	0	1	11	50
10	6	10	4.7	54	254	2	2½	5	4	5	1	2	2	65
12	6	10	4.7	54	254	2½	3	6	5	6	2	3	1	65
12	7	10	6.4	54	346	2½	3	6	6	6	2	3	0	90
14	7	10	6.4	54	346	2½	3	6	6	6	2	3	1	90
12	7½	10	7.39	54	399	2½	3	6	6	6	2	3	0	100
14	7½	10	7.39	54	399	2½	3	6	6	6	2	3	1	100
16	8	10	8.44	54	455	2½	3	6	6	6	7	3	10	105
12	8½	10	9.56	54	516	2½	3	6	6	6	7	3	0	130
14	8½	10	9.56	54	516	2½	3	6	6	6	7	3	1	130
16	8½	10	9.56	54	516	2½	3	6	6	6	7	3	10	130
14	9¼	10	11.37	54	614	2½	3	8	7	6	7	3	1	150
16	9¼	10	11.37	54	614	2½	3	8	7	6	8	3	10	150

An additional charge is made for Tobin-bronze piston rods, brass water pistons, bed plates or for any extras.

The 8½-inch and the 9¼-inch water ends can be fitted with 18½-inch or 20-inch steam cylinders, if desired.

and less length will be required for the run up and down the pile. The line for raising the jets is connected to the U at the top of the jet, run over a sheave on the outside of the head block and down to the nigger head on the engine.

Where deep jetting is to be done as mentioned in the foundations of the Eleventh Street bridge at Tacoma, Chapter X, these precautions are doubly important, to save every pound of pressure that might be lost by an unduly long jet line from the pump to the nozzle. The loss of pressure per hundred feet of hose and pipe may be found from Table XI, and can be greatly reduced by using the largest sized pipe and hose up to the jet that can be easily handled. The importance of avoiding loss of pressure may be realized by figuring out the back pressure from the depth to jet the piles, which in the 160 feet below high water in the above case,

TABLE VI.—PACKED PLUNGER PUMP FOR 200 POUNDS PRESSURE.

Diameter of Steam Cylinders, Inches.	Diameter of Water Plungers, Inches.	Length of Stroke, Inches.	Gallons per Revolution.	Revolutions per Minute.	Gallons per Minute.	Sizes of Pipes for Short Lengths to be Increased as Length Increases.				Approximate Space Occupied Feet and Inches.		Size Boilers Required for very Heavy Work. H.P.
						Steam Pipe, In.	Exhaust Pipe, In.	Suction Pipe, In.	Delivery Pipe, In.	Length.	Width.	
10	6	10	4.64	45	220	2	2½	5	4	7 7	2 6	85
12	7	10	6.4	45	300	2½	3	6	5	8	3	115
14	7	10	6.4	45	300	2½	3	6	5	8	3 1	115
14	8½	10	9.56	45	442	2½	3	6	5	8 4	3 2	165
16	8½	10	9.56	45	442	2½	3	6	5	8 4	3 10	165
18½	8½	10	9.56	45	442	3	3½	6	5	8 5	4	165
14	10¼	10	13.95	45	645	2½	3	8	7	8 6	3 6	240
16	10¼	10	13.95	45	645	2½	3	8	7	8 7	3 8½	240
18½	10¼	10	13.95	45	645	3	3½	8	7	8 8	4	240
16	12	10	19.16	45	830	2½	3	10	8	8 7	3 10	330
18½	12	10	19.16	45	830	3	3½	10	8	8 8	4	330
14	8½	15	14.14	40	590	2½	3	6	5	9 5	3 7	225
17	8½	15	14.14	40	590	2½	3½	6	5	9 6	3 11	225
17	10¼	15	20.83	40	855	2½	3½	8	7	10 1	3 11	325
20	10¼	15	20.83	40	855	4	5	8	7	10 2	4 2	325
17	12	15	28.78	40	1175	2½	3½	10	8	10 2	3 11	450
20	12	15	28.78	40	1175	4	5	10	8	10 4	4 2	450

An additional charge is made when the pumps are fitted with Tobin-bronze piston rods and brass plungers with brass-bushed plunger stuffing boxes.

amounted to 69 pounds per square inch, and with 300 feet of pipe and hose the resultant pressure from a pump giving 200 pounds pressure at the pump, would only be from 130 to 140 pounds per square inch at the nozzle, and with a poor arrangement of piping, and hose in bad condition, the pressure at the nozzle might run much below 100 pounds. Some engineers, losing sight of the fact that only a fixed amount of water will go through the jets at a fixed pump pressure, will order larger pumps put on when the jets are not effective, but what is needed in such a case is almost always a greater pump pressure. Where the material is loose gravel or an open boulder bed,

TABLE VII.—COMPOUND PACKED PLUNGER PUMP FOR 200 POUNDS PRESSURE.

Diameter of Steam Cylinders, Inches.	Diameter of Water Plungers, Inches.	Length of Stroke, Inches.	Gallons per Revolution.	Revolutions per Minute.	Gallons per Minute.	Sizes of Pipes for Short Lengths to be Increased as Length Increases.				Approximate Space Occupied Feet and Inches.		Size Boilers Required for very Heavy Work. H.P.
						Steam Pipe, In.	Exhaust Pipe, In.	Suction Pipe, In.	Delivery Pipe, In.	Length.	Width.	
8 & 12	7	10	6.68	45	300	2	3	6	5	9 9	3 1½	90
9 & 14	7	10	6.68	45	300	2	3	6	5	9 10½	3 1½	90
10 & 16	7	10	6.68	45	300	2	3	6	5	10	3 9½	90
12 & 18½	7	10	6.68	45	300	2½	3½	6	5	10	4	90
9 & 14	8½	10	9.8	45	442	2	3	6	5	10 1½	3 1½	135
10 & 16	8½	10	9.8	45	442	2	3	6	5	10 3	3 9½	135
12 & 18½	8½	10	9.8	45	442	2½	3½	6	5	10 3	4	135
14 & 20	8½	10	9.8	45	442	2½	5	6	5	10 1½	4 1⅜	135
10 & 16	10¼	10	14.3	45	645	2	3	8	7	10 6½	3 9½	195
12 & 18½	10¼	10	14.3	45	645	2½	3½	8	7	10 6½	4	195
14 & 20	10¼	10	14.3	45	645	2½	5	8	7	10 5	4 1⅜	195
10 & 16	12	10	19.6	45	880	2	3	12	10	10 6	3 9½	270
12 & 18½	12	10	19.6	45	880	2½	3½	12	10	10 6	4	270
14 & 20	12	10	19.6	45	880	2½	5	12	10	10 4½	4 1⅜	270
9 & 14	8½	15	14.7	40	590	2	3	6	5	10 10	3 6¾	180
12 & 17	8½	15	14.7	40	590	2½	3½	6	5	11 9	4	180
14 & 20	8½	15	14.7	40	590	2½	5	6	5	11 9½	4 1¼	180
12 & 17	10¼	15	21.4	40	855	2½	3½	8	7	12	4	255
14 & 20	10¼	15	21.4	40	855	2½	5	8	7	12 1	4 1⅜	255
12 & 17	12	15	29.4	40	1175	2½	3½	12	10	12	4	360
14 & 20	12	15	29.4	40	1175	2½	5	12	10	12 1	4 1⅜	360

An additional charge is made when pumps are fitted with Tobin-bronze piston rods and brass plungers with brass-bushed plunger stuffing boxes.

TABLE VIII.—LOCOMOTIVE BOILERS WITH WATER BOTTOM.

Nominal Rated Horse Power.	Shell.		Fire Box.			Dome.		Size of Steam Outlet. In.	Tubes.			Total Heating Surface Sq. Ft.	Stack.		Shipping Weight. Pounds Approximate.		
	Diam. In.	Length over all Ft. In.	Length. In.	Width. In.	Height. In.	Diam. In.	Height. In.		No.	Diam. In.	Length. Ft. In.		Diam. In.	Length. Ft.	Boiler only.	Fixtures only.	Boiler and Fixtures.
10	28	9 5	30	23	36	15	16	1½	25	2½	5 6	127	10	15	2754	446	3200
15	32	11 0	42	27	40	15	16	2	34	2½	6 0	188	12	15	3829	671	4500
20	34	12 8	48	29	45	15	16	2	40	2½	7 0	251	12	20	4902	798	5700
25	36	14 0	52	32	48	18	20	2½	45	2½	8 0	315	14	25	5832	968	6800
30	40	15 2	54	36	48	20	22	2½	49	2½	9 0	376	18	35	6636	1364	8000
40	42	16 4	54	37	60	20	22	3	62	2½	10 0	510	20	40	8445	1555	10000

Each boiler, on skids, is furnished with dome, grate bars, safety valve, steam gage and siphon, glass water gage and gage cocks, check valve, stop valve blow-off cock, whistle, smock stack, and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

TABLE IX.—LOCOMOTIVE BOILERS WITH OPEN BOTTOM.

Nominal Rated Horse Power.	Shell.		Fire Box.			Dome.		Size of Steam Outlet. In.	Tubes.			Total Heating Surface. Sq. Ft.	Stack.		Shipping Weight. Pounds Approximate.		
	Diam. In.	Length. Over all Ft. In.	Length. In.	Width. In.	Height. In.	Diam. In.	Height. In.		No.	Diam. In.	Length. Ft.		Diam. In.	Length. Ft.	Boiler only.	Fixtures. only	Boiler and Fixtures.
50	50	17 6	60	44	46	24	24	3	60	3	11	600	22	40	9500	2000	11500
60	52	18 6	60	46	48	28	26	3½	68	3	12	725	24	40	10922	2078	13000
70	54	19 6	60	48	50	28	26	3½	76	3	13	870	26	40	12670	2330	15000

Each boiler, on skids, is furnished with dome, grate bars, safety valve, steam gage and siphon, glass water gage, and gage cocks, check valve, stop valve, blow-off cock, whistle, smoke stack and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

TABLE X.—INTERNAL FIRED BOILERS WITH INDEPENDENT DOME.

Nominal Rated Horse Power.	Shell.			Furnace.		Mean Thickness.			Dome.		Size of Steam Outlet. In.	Return Tubes.			Total Heating Surface. Sq. Ft.	Stack.		Shipping Weight. Pounds Approximate.		
	Diam. In.	Length over all Ft.	In.	Diam. In.	Length. In.	Shell. In.	Heads. In.	Flue. In.	Diam. In.	Height. In.		No.	Diam. In.	Length. Ft.		Diam. In.	Length Ft.	Boiler only.	Fixtures only	Boiler and Fixtures.
10	40	9	7	20	30	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{3}{8}$	15	16	$1\frac{1}{2}$	16	3	8	138	12	25	4529	471	5000
15	46	9	7	24	40	$\frac{1}{4}$	$\frac{7}{16}$	$\frac{3}{8}$	15	16	2	24	3	8	196	14	25	5513	687	6200
20	52	9	7	26	46	$\frac{9}{32}$	$\frac{7}{16}$	$\frac{13}{32}$	15	16	2	30	3	8	240	16	25	6621	779	7400
25	56	9	7	28	50	$\frac{9}{32}$	$\frac{1}{2}$	$\frac{13}{32}$	18	20	$2\frac{1}{2}$	38	3	8	300	18	30	7555	1045	8600
30	60	9	7	30	50	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{7}{16}$	20	22	$2\frac{1}{2}$	47	3	8	362	20	30	8818	1182	10000
40	66	9	8	34	56	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	24	24	3	68	4	8	496	24	30	10513	1387	12100

Each boiler, on skids, is furnished with dome, grate bars, safety valve, steam gage and siphon, glass water gage, and gage cocks, check valve, stop valve, blow-off cock, whistle, smoke stack and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

TABLE XI.—FRICTION LOSS IN JET PIPES AND HOSE, AVERAGE SMOOTHNESS

Diameter.	Gallons per Minute Flowing.																						
	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000	1050	1100	1150	1200
	Friction Loss, Pounds per 100 Ft. Length																						
2	14	30	55	85	120																		
2½	5	10	18	28	40	54	72	90	112														
3	2	4	7	12	16	22	30	40	45	54	65	76	88	100	115								
3½	..	2	3	5	8	10	13	16	20	24	30	34	41	45	53	58	65	72	83	88	97	106	115
4	..	1	2	3	4	5	7	8	11	12	15	17	21	23	28	30	33	36	40	44	4	52	56
5	..	..	..	..	1	2	2	3	4	4	5	6	7	8	9	10	11	13	14	15	17	18	20
6	..	..	..	..	..	..	..	1	1	2	2	2	3	3	4	4	5	5	6	6	7	7	8
7	..	..	..	..	..	..	..	..	..	1	1	1	1	1	2	2	2	2	3	3	3	3	4
8	..	..	..	..	..	..	..	..	..	...	...	...	...	...	1	1	1	1	1	2	2	2	2

Values were taken from a diagram, to the nearest pound. Table shows that for large-sized pipe, friction loss may be practically disregarded, within the capacities of pumps up to 8 inches discharge.

the water will dissipate through the bottom and fail to lubricate the sides of the pile and the jetting plant be almost valueless, and the use of larger pumps be of little avail, unless of course the hose, piping and nozzle are made very much larger so as to actually deliver a much larger volume of water to overcome the seepage. The largest pipe that can be reasonably handled for the jets themselves is 3 inches in diameter, and the nozzle could be increased to 1½ inches or 1¾ inches. But for compact material where the water will not be lost, and for depths out of the ordinary, it is pressure that is needed, and the required pressure to accomplish the desired results may be obtained by a careful consideration of the conditions and a common-sense layout of the plant and piping. The required pump pressure may be taken from the fire stream data as given in Table XII, for various sized nozzles.

TABLE XII.—DISCHARGE IN GALLONS PER MINUTE FROM NOZZLES ATTACHED TO 50 FT. OF 2½-IN. SMOOTH-LINED HOSE.

Pressure at Pump.	Size of Smooth Nozzle.							
	1¾	1½	1⅜	1¼	1⅛	1	⅞	¾
10	195	165	145	125	105	85	70	50
20	275	230	205	180	150	125	95	70
30	335	285	250	220	185	150	120	90
40	390	325	290	255	215	175	135	100
50	430	365	325	285	240	195	150	115
60	475	400	355	310	260	215	165	125
70	510	430	385	335	280	230	180	135
80	545	460	410	355	300	245	190	145
90	580	490	435	380	320	260	205	150
100	610	515	460	400	335	275	215	160
110	640	540	480	420	350	290	225	170
120	670	570	505	440	370	300	235	175
130	700	590	530	455	385	315	245	180
140	730	615	550	475	400	330	250	190
150	760	640	570	495	420	340	260	195
160	790	660	590	510	430	350	270	200
170	820	680	610	525	445	360	280	205
180	850	700	630	545	460	370	285	210
190	880	720	650	560	475	380	290	215
200	910	740	670	580	490	395	300	220

Values read from diagram to nearest five pounds.

Unless the borings made in the original soundings for the foundations prove to be reliable, and from which it is presumed the pumping plant will be designed, much experimenting will have to be done to

arrive at the best arrangements that can be made, and should changes be found necessary the bottom should be sounded thoroughly at the site of each pier with the jets, which will best develop the actual conditions to be met with. In some instances good results have been obtained in a somewhat porous bottom, by using a large pump and perforating the sides of the jet pipe every few feet with ⅛-inch holes. This can only be done at the bottom end, however, as the water must be shut off before these holes come above the surface of the water to avoid a deluge over the men and plant, and shutting off the water is dangerous, as the jets are then almost sure to freeze or stick in the bottom, so they cannot be pulled out, and are thus lost for future use.

TABLE XIII.—HEAD IN FEET WITH EQUIVALENT IN POUNDS PRESSURE PER SQUARE INCH.

Head	Pressure	Head	Pressure	Head	Pressure	Head	Pressure	Head	Pressure
1	.434	29	12.58	57	24.73	85	36.89	165	71.61
2	.868	30	13.02	58	25.17	86	37.32	170	73.78
3	1.30	31	13.45	59	25.60	87	37.75	175	76.90
4	1.73	32	13.88	60	26.04	88	38.29	180	78.12
5	2.17	33	14.32	61	26.47	89	38.62	185	80.29
6	2.50	34	14.75	62	26.90	90	39.06	190	82.46
7	3.03	35	15.19	63	27.34	91	39.49	195	84.63
8	3.47	36	15.62	64	27.76	92	39.92	200	86.80
9	3.90	37	16.05	65	28.21	93	40.36	210	91.14
10	4.34	38	16.49	66	28.64	94	40.79	220	95.48
11	4.77	39	16.92	67	29.07	95	41.23	230	99.85
12	5.20	40	17.36	68	29.51	96	41.66	240	104.16
13	5.65	41	17.79	69	29.94	97	42.09	250	108.50
14	6.07	42	18.22	70	30.38	98	42.53	260	112.84
15	6.51	43	18.66	71	30.81	99	42.96	270	117.66
16	6.94	44	19.09	72	31.24	100	43.40	280	121.52
17	7.37	45	19.53	73	31.68	105	45.57	290	125.86
18	7.81	46	19.94	74	32.11	110	47.74	300	130.50
19	8.24	47	20.39	75	32.55	115	50.91	350	152.20
20	8.68	48	20.83	76	32.98	120	52.08	400	173.60
21	9.11	49	21.26	77	33.41	125	54.25	450	195.30
22	9.54	50	21.70	78	33.85	130	56.45	500	217.00
23	9.88	51	22.17	79	34.28	135	58.62	600	260.40
24	10.41	52	22.56	80	34.72	140	60.76	700	303.80
25	10.85	53	22.80	81	35.15	145	63.93	800	347.20
26	11.06	54	23.43	82	35.58	150	65.10	900	390.60
27	11.71	55	23.87	83	36.02	155	67.27	1000	434.00
28	12.15	56	24.30	84	36.45	160	69.44	1500	651.00

The use of jets either from a pile-driver scow or a pump scow (Fig. 70) is the most satisfactory, as it employs a low lift for the

pumps, gives a short delivery line, which reduces the losses at both ends, and results in a much greater pressure at the nozzle than is the case with a pumping plant located in one position on shore or on falsework. The jets should always be run down into the bottom, as has been stated, and then the pile dropped in, the jets kept going up and down the pile as it is tapped with the hammer and driven home.

The same pump scow should be rigged to handle the sand pumps or hydraulic elevators on a piece of work similar to that described in Chapter X, an extra pump being provided not less than $10 \times 6 \times 10$ to supply a jet to loosen up the material for the pump or elevator. Whether the sand pump is used for excavating the crib or not, it is useful in cleaning out for concreting, and especially necessary where piles have been driven in the crib, to remove the material swelled up in driving and jetting the piles. While it is usual to dredge out a crib somewhat deeper than required to allow for this, the jetting will often fill up a crib from 4 feet to 10 feet with soft stuff which must be removed to have the bottom in proper condition to receive the concrete.

Where cribs are to be sunk to great depths, it is advisable to have a pumping plant to supply water to jets for jetting around the crib to reduce the friction. This can either be done by separate jets or by jets built into the crib or caisson, and this might require a much larger equipment of boilers and pumps than would be needed for the jetting of piles.

The depth to which it is necessary to jet piling in a bridge foundation is determined from one of three conditions, first, to get below scour; second, to reach a hard substratum which is to carry the load; and third, to a sufficient depth in a soft bottom so that the frictional resistance of the pile will carry the required load.

The depth to go in the first case must be arrived at from a careful study of the location as to the depth scour will reach, bearing in mind the changed flow that will result in placing piers in the river. If there is any question about this, the piles must be given a penetration of a sufficient amount to make the piers safe beyond question, and riprap must be placed around the pier, so as to prevent scour starting.

The probable result of such study will doubtless indicate that the crib or caisson must be carried deeper, as scour below the cutting edge would lay the piles bare and make the safety of the pier doubtful.

The second case, where piles are required to be jetted down to

reach a hard substratum, would cause the piles to act as timber columns, and consequently would be very much limited as to the safe or economical depth. Probably not over 40 to 50 feet would be considered safe for the ordinary-sized piling, for the ordinary loads.

If piles are considered as columns, the figures would sometimes indicate that they would carry a greater load than indicated by the friction, or the usual formula $\frac{2wh}{s+1}$, so they must be used with very careful judgment, especially as the pile in reaching hard bottom may rest on the point only.

Where the engineer can be sure that the material through which the piles have been driven to the hard bottom is firm enough to give sidewise support to the pile, they can be figured as short columns (Table III, Chapter IV) to carry a load of 1000 pounds per square inch as given by Rankine, or the lesser value of 786 pounds as given by Peronnet for ordinary, good timber. The values for various classes of timber as shown by recent tests are given in the tables of timber in Chapter XXVIII.

The depth to go in the third case where the friction on the piles must carry the load, is to be determined by applying the formula for driving with a drop-hammer or a steam hammer as the case may be, until the pile is driving hard enough to carry the load. Where the piles are being jetted, it is the usual custom to stop jetting at about the required depth, and hammer the pile for some little time to see what the penetration per blow is after getting below the material disturbed by the jet, in order to apply the formula.

In many cases where piles have been jetted and allowed to set for some hours until the material has frozen around them, they cannot afterwards be moved with the hammer, or at least very little penetration will result, which fact goes to show that in many cases piles are driven to ridiculous depths simply because the jets will assist them there. In no case should they be jetted deeper than necessary to carry the load by the friction on the surface, as set forth in Chapter IV, that is, 500 pounds per square foot of surface in firm sand or clay, or as low as 120 pounds per square foot in silt. Careful actual tests in any case where a large number of piles are to be driven might indicate that these values could be doubled or trebled, with perfect safety.

CHAPTER VI

CONSTRUCTION WITH SHEET-PILES *

WATER pressure against the sides of a sheet-pile coffer-dam is seldom provided for in an accurate manner, the thickness of the piling being usually decided upon from past experience, as are also the size and spacing of the guide-piles and wales.

These are points where guess-work should be eliminated, as otherwise good coffer-dams are often seen where the pressure has so bulged the plank as to cause leakage. While this may perhaps be corrected by additional bracing, simple calculations may easily be made to determine the size beforehand.

The pressure against a coffer-dam may act as at (*a*), Fig. 80, the sheet-piling being in the condition of a beam fixed at one end and loaded with a gradually increasing weight, as shown by the dotted lines, due to the pressure of water or puddle at 62.4 pounds per cubic foot. Then the load on a width w of the wall is $124.8wd^2$ and the moment of the pressure is $83.2wd^3$. Taking the allowable unit stress on wet timber at 1000 pounds per square inch, the thickness t of the sheet-piling may be obtained from the formula

$$t=\sqrt{.496d^3},$$

in which d is to be taken in feet, and the resulting value of t will be the thickness in inches of the sheet-piling.

This formula has been expressed in a graphic manner in diagram (*d*), Fig. 80, from which, knowing the depth of water $2d$. the thickness of piling may be read directly without calculation.

The addition of a strut, as at (*b*), Fig. 80, places the sheet-piling in the condition of a beam supported at the upper end and fixed at

* The assumption that the pressure of puddle will be the same as water pressure is made advisedly. It is true that *very* wet clay, approaching a fluid condition, will exert a much greater pressure, but it would then be useless as puddle. Dry clay would exert a pressure of less than half that due to water, so it has been assumed that *wet* clay or puddle would exert the same force as water. Should it exceed it for a short time no damage would be done, owing to the low unit stress adopted.

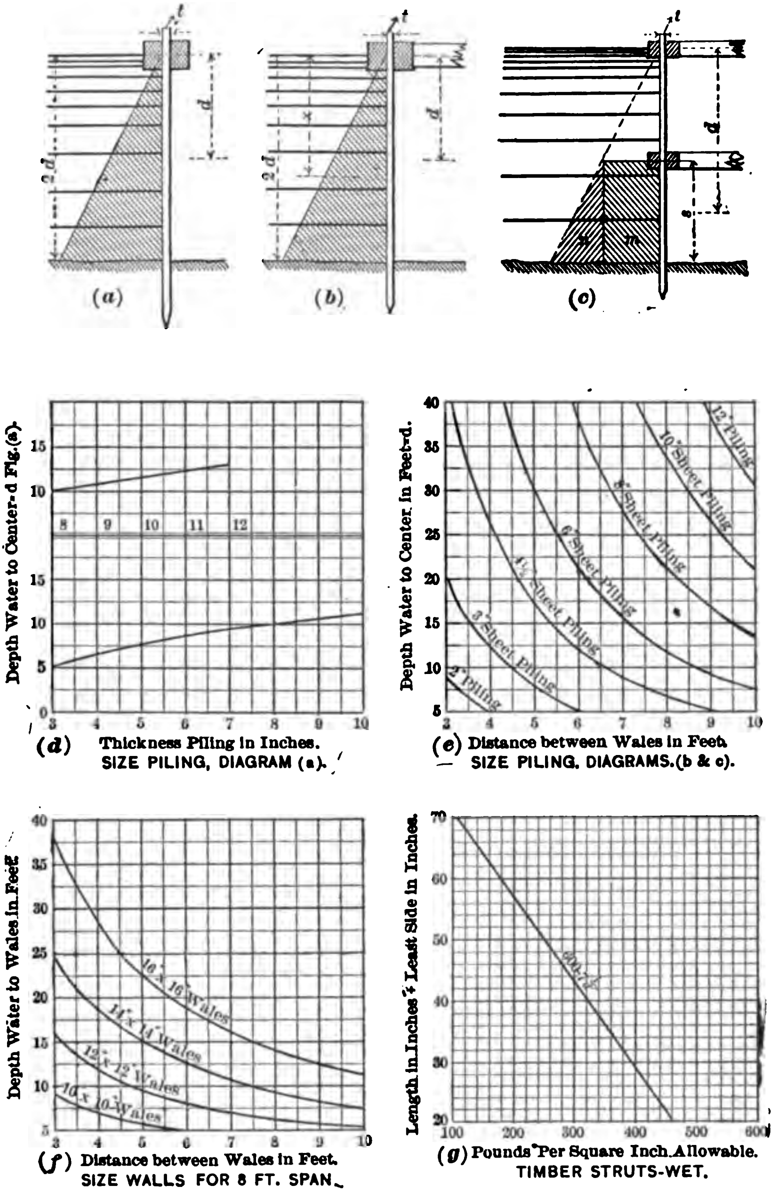


FIG. 80.—ARRANGEMENT AND DIAGRAMS OF SIZES FOR SHEET-PILE COFFER-DAMS.

the lower end, but for practical reasons it is best to consider it as merely supported at both ends. The load will be the same as in the former case, $124.8wd^2$, but the maximum moment will occur at a point x, which is a distance from the top equal to 1.16 times d, and has a value of $32wd^3$. The thickness t may be found from the formula

$$t=\sqrt{.192a^3}.$$

When the section of the plank to be calculated is located as "s" in (c) of Fig. 80, it is in the condition of a beam fixed at both ends and loaded with a uniform load m and a triangular load n. The exact analysis of this is too lengthy to be taken up here, and reference may be made to page 195 of Wood's "Resistance of Materials."

For practical purposes we may consider the load as all uniform and due to the head acting at the middle of the span. This will give a load of $62.4wds$ on the span s for a width w, and a moment of $7.8ds^2$, which gives a formula for practical use, for a unit stress of 1000 pounds per square inch of

$$t=\sqrt{.047ds^2}.$$

This is closely represented graphically in diagram (e) of Fig. 80 which may also be used for case (b) by taking the depth of water to the middle of the span. For example, when the depth of water to the middle of the span is 15 feet, find this in the vertical column to the left, and if 6-inch sheet-piles are to be used, follow the horizontal through 15 feet until it intersects the 6-inch curve and vertically beneath will be found the maximum spacing of wales, 7 feet 3 inches.

The size and spacing of wales may be taken from a similar diagram (f) of Fig. 80, which assumes the guide-piles to be 8 feet apart. The spacing of struts or braces will vary so much that the load must be calculated, and when this and the length are known the size may be calculated from diagram (g) of Fig. 80, which is for wet timber.

From the formula

$$p=600-7(l\div d),$$

in which p is the allowable stress in pounds per square inch, l is the unsupported length in inches, and d the least side of the stick in inches.

Where two rows of sheet-piling are to be driven to form a puddle-chamber, if they are to be efficiently braced from the inside of the coffer-dam, it will be sufficient to have a thickness of puddle of from

2 to 4 feet to exclude the water, depending on the quality of the puddle. Where there is to be no internal bracing, but two rows of sheet-piling braced together and filled with puddle are to resist overturning, the common rule is to make the width of the puddle-chamber equal to the height above ground, up to 10 feet. When the height exceeds 10 feet, add one-third to the excess height to 10 feet for the width.

When the puddle-chamber becomes very wide it is often divided into several compartments, as was shown in Fig. 5, and stepped in a similar manner. When the bottom is rock overlaid with a thin deposit of clay or gravel, the sheet-piles may be driven around an open crib-work for support, as was done at Harper's Ferry, on the B. & O. R. R.

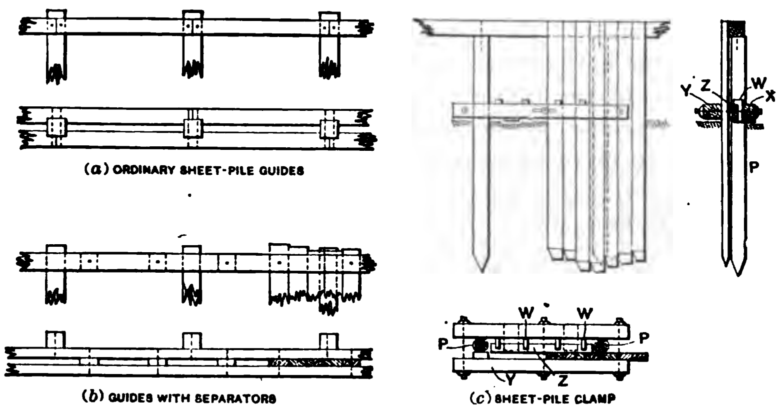


FIG. 81.—SHEET-PILE GUIDES AND CLAMPS.

Where the guide-piles are to be used, the waling-pieces are framed in, as was specified on the Hutcheson Bridge, as shown at (*a*), Fig. 81, where the guide-piles are of sawed timber. The wales are spaced slightly farther apart than the thickness of the sheet-piles, to allow clearance in driving, the space between the guide-piles being filled out with a key pile to fill the panel tightly. This method is but little used with tight piling, that shown at (*b*), Fig. 81, allowing the piling to be driven continuously, by removing the spacing blocks as they are reached, and substituting bolts through the sheet-piles, firmly connecting the piles and wales together.

A very satisfactory method is described in the *Engineering News* of May 12, 1892, which was used by A. F. Walker. Having occasion to do a large amount of work it was desirable not to go to the expense

of squared guide-piles. Round guide-piles (*P*) were first driven 7 feet apart, and cut off to a level. Caps were then drift-bolted to the tops, allowing them to project slightly beyond the face of the round piles, thus forming a permanent support for the top of the sheet-piles. Near the ground line was placed the clamp, consisting of two sticks (*X*) and (*Y*), connected by three bolts and drawn together as tight as the intervening piles or pile and gage-block (*G*) will permit. The stick (*Z*) is then forced forward by the wedges (*W*) until the space between (*Z*) and (*Y*) is the same as the thickness of the piles. The pieces (*X*), (*Y*), (*Z*) are slotted for the middle bolt, and this permits of some adjustment. When one of the piles partially closes this slot, a notch is cut in it large enough to receive the bolt, and the bolt is then slipped up to it and tightened. This allows of the next pile being driven as close as the others. When one panel has been completed the nuts are removed and the clamps moved forward to the next one, a notch being cut in the end pile to receive the end bolt of the clamp. The piles are sharpened flatwise with a little more slope on the side facing the guide-piles, giving them a tendency to drive away from the guide-pile at the foot and bear against the cap at the top. A slight bevel is also given to the edge to make the foot crowd the adjoining pile. During the first half of the driving, the joint is held a little open at the top, but during the latter half, pressure is brought to crowd it toward its neighbor, and the joint will close as tightly as possible.

The use of single pieces of timber as wales, against which the sheet-piling is driven, is illustrated in the use of method (*b*) of Fig. 52, by Benj. Douglas, bridge engineer of the Michigan Central Railway. The coffer-dam (Fig. 82) was built without guide-piles, the wales being 12×12-inch timber bolted against the outside of the sheet-piling, by the brace rods 1 inch in diameter. The wales are held in place vertically by bracing of 2×12-inch pine plank, which are spiked on as verticals and diagonals to form a truss and also to stiffen the framework in general.

The sheet-piling is 6×12, and after being driven into the hard gravel bottom, the cracks were lapped by 1-inch boards. The bottom was uneven and accounts for the difference in height, the excavation at the high end being dumped outside at the low end, to assist in making the dam tight. The puddle-chamber was 2 feet 8 inches wide and was filled with clayey gravel. The plan also shows the grillage in place for receiving the foundation courses of the stone-work. This is formed by 12×12 timber crossed, and drift-bolted together with 1-inch round and 18-inch long drift-bolts.

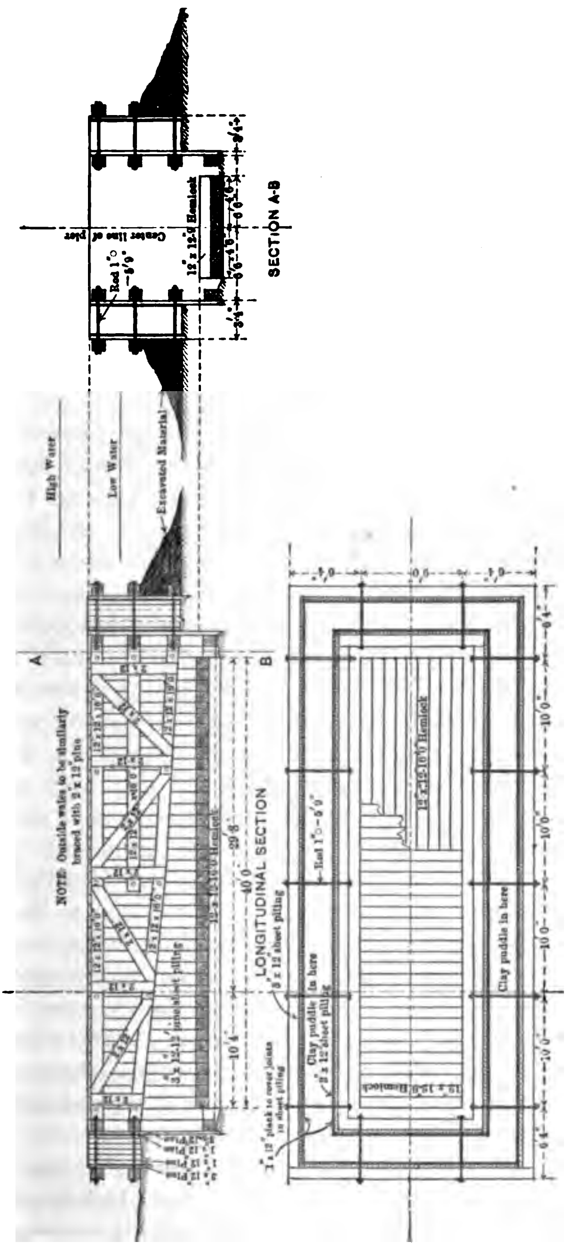


FIG. 82.—COFFER-DAM FOR ANN ARBOR BRIDGE. MICHIGAN CENTRAL RAILWAY.

The account of the Arthur Kill Bridge foundation in Vol. 27 of the "Transactions of the American Society of Civil Engineers," by A. P. Boller, consulting engineer, covers a very interesting experience with sheet-piling on pier No. 5: "This pier is near the edge of the marsh forming the Staten Island shore, which is barely flooded at extreme high tides. Borings indicated about 30 feet from the surface to hard bottom, consisting of mud, mud and clay, clay and shale to the bottom of shaley clay, in which the pier was to be founded. Experience on other work of a similar character indicated that the founding of this pier would be accomplished with little difficulty. The area of the foundations was inclosed with a tongued-and-grooved sheet-pile dam of 4-inch yellow pine plank. But it was found impossible to hold the plank at a depth of 15 feet, the mud and clay becoming puddled with water, and despite all efforts at bracing, the plank shoved inward to such an extent as to spoil the whole dam before we were half way down. A second dam was therefore driven around the first one, but this time with 10×12-inch tongued-and-grooved timbers, in one length to reach the extreme bottom. These timbers were grooved by slitting the grooves out at the mill with a circular saw, and chiseling the blank so formed free. The tongue was an independent spline, $2\frac{1}{2}\times4$ inches, of dry wood and nailed in one groove. The timbers were shaped at the feet to drive close. This dam was hard driving but was finally accomplished, when digging was resumed and the old dam removed piecemeal as we could get in the braces. The bottom was reached within a perfect dam, with only one bad leak in the northwest corner, due to the shattering of a small piece of one tongue during the driving. As it was impossible to stop this leak from the inside, and the outside was inaccessible, to prevent washing the concrete, the leak was led off in a box at the side of the dam to the sump-well, and the footing course of concrete, filling the whole area of the dam about seven feet deep, was gotten in place."

This example emphasizes in a very decided manner many of the statements that have been made heretofore. While no doubt the removal of the old dam was attended with much expense, its inclosure entirely within the new sheet-piling rendered the prosecution of the work comparatively certain.

An example of the driving of sheet-piling on a slant, to prevent crowding in at the bottom, is shown in Fig. 83, which is a cross-section of a sewer coffer-dam used on the Metropolitan Sewerage Systems of Massachusetts by Howard A. Carson, chief engineer, and described in the *Engineering News* of Feb. 8, 1894.

The outlet into the ocean at Deer Island begins at a point about 60 feet inside the high-water line, and about 1850 lineal feet is from 5 to 10 feet below high water. This necessitated the coffer-dam, which was constructed with bents every 6 feet and with 2-inch plank inside the high water-line, but for the remaining distance of 4-inch matched plank. The excavation was done by means of buckets, traveling-derricks, and dump-cars, the latter being emptied at the sides and ends of the trench. The leakage from the ocean was kept out by using centrifugal pumps, which pumped a maximum of 46,000 gallons per hour. The concrete, which has large boulders embedded in its surface the size of paving-stones, was carried up to the level of the ocean bottom.

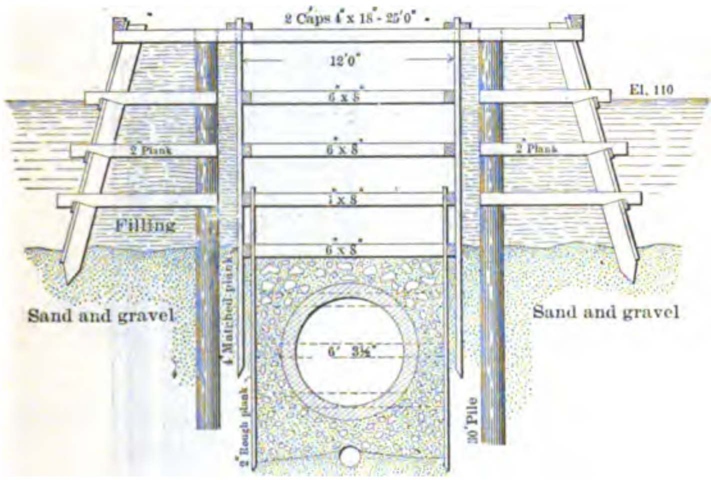


FIG. 83.—SEWER COFFER-DAM. BOSTON SEWERAGE SYSTEM.

From the middle of June, 1893, when the work was begun, to the end of September, 526 feet of trench were completed. The size of the trench was 14 feet average depth and 10.8 feet average width, which made the exacavation average 5.6 yards per lineal foot. The cost of the trench, including coffer-dam, sheeting left in, and back-filling, was $44 per lineal foot.

Casual mention has been made in several places of the use of Wakefield sheet-piling which was illustrated at *h* and *h'* of Fig. 52 and which is further shown in Fig. 84. View No. 1 is of a corner which is formed as in the plan No. 2, a tongue being bolted on the side of a pile, when the corner is reached as in No. 3. Any angle is turned by a similar method, which is shown by No. 4, or the piles

may be driven to form a curve. The essential features of the system are the triple lap or long tongue and groove which excludes the

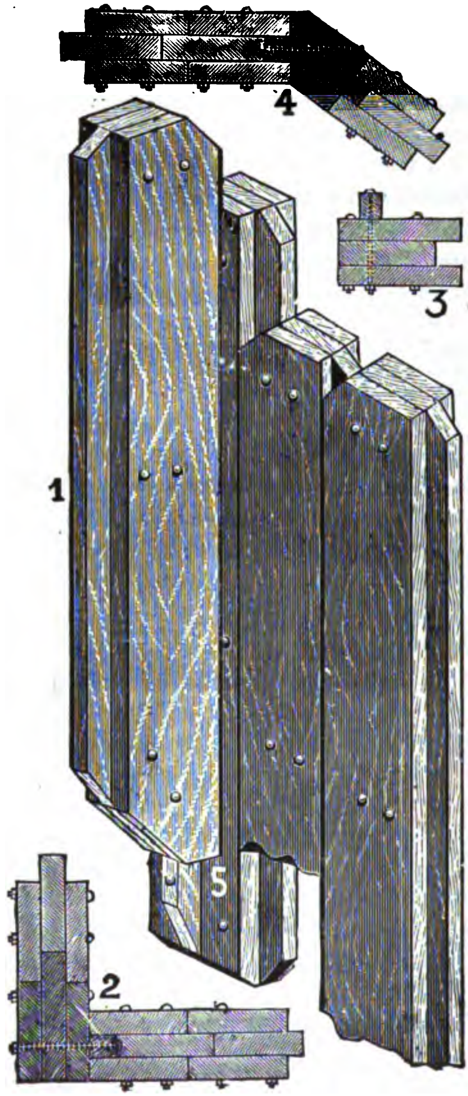


FIG. 84.—WAKEFIELD SHEET-PILING.

water, and the use of ordinary plank, which can be easily obtained. The center plank should be sized to a uniform thickness, to insure the tongues fitting the grooves, and to make driving easy, while the

three plank are to be bolted and spiked together to cause them to act as a compound beam and not as separate plank like the system of (*b*), Fig. 52. It is recommended to use a 2½-inch tongue on 1-inch boards and ⅜-inch bolts. For 1½-inch plank a 3-inch tongue, for 2-inch and 2½-inch plank a 3½-inch tongue and ½-inch bolts, while for 3-inch plank a 3½-inch tongue and ⅝-inch bolts are to be used, and the same size bolts for 4-inch plank, but a 4-inch tongue. Two bolts are to be staggered in every 5 to 8 feet of the length of the pile, and spikes used between the bolts on long piles.

The La Grange lock on the Illinois River was inclosed with this piling, under the direction of Major W. L. Marshall, Corps of Engineers. It was intended to back the sheeting with earth, but as both dredges broke down the water-tightness was entirely dependent on the Wakefield piling, and under a 7-feet head no leaks were developed. The piles were made of three plank 3×12 inches by

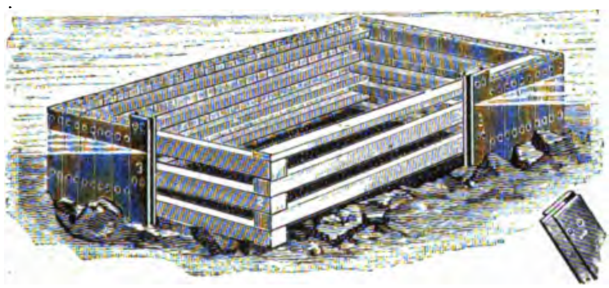


FIG. 85.—TYPE OF MOMENCE AND HARPER'S FERRY COFFER-DAMS.

22 feet long and with a 3-inch tongue; they were driven by three pile-drivers with hammers of from 2800 to 3000 pounds through sand and mud, and in one place a layer of shells. There was no difficulty experienced in driving the piles without special appliances.

The use of 1-inch boards in this form (Fig. 85) is described by H. F. Baldwin, chief engineer of the C. & E. I. Railway: "In constructing our second track over the Kankakee River at Momence, Ill., it was necessary to extend the piers in that river. The bottom is limestone and the surface is very irregular. We tried several days and finally succeeded in constructing a coffer-dam with two parallel walls of sheet-piling. We then tried the Wakefield triple lap-piling, constructed of 1-inch boards sharpened to an edge, 2½ tongue and groove, which were driven with sledges until the piles, which were soft pine, conformed to the uneven surface of the rock. This piling was driven around cribs loaded with stone, and, after the piling

was driven, gravel was put outside the coffer-dam, after which no trouble was experienced in pumping out the water."

The work on the foundations of the new B. & O. R. R. bridge over the Potomac River at Harper's Ferry was similar in many respects to the above, and the system was found to be very satisfactory.

References were made to the use of this piling on the Charlestown Bridge at Boston and the driving of the piles shown in Fig. 53. The work was under the charge of Jno. E. Cheney, consulting engineer,

FIG. 86.—COFFER-DAM ON CHARLESTOWN BRIDGE.

and was successfully carried out. The piling were driven principally as forms for concrete foundations, and but little care was taken to make the dams water-tight. After the concrete was deposited they were used as coffer-dams against a 6- or 7-feet head of water. They were 18 feet 6 inches by 119 feet (Fig. 86) and in some cases were 30 feet below low water or 40 feet below mean high water. The piling was made of 2-inch plank and driven with an ordinary pile-driver. The pumping was done with a 20-inch centrifugal pump, and in some cases a 12-inch Follansbee pump of the propeller type was used.

The construction of the sewerage system at Fort Monroe, Va.,

under Capt. Thos. L. Casey, Corps of Engineers, is described in the report of the Chief of Engineers of 1896. The work was done on the general plans of Rudolph Hering, consulting sanitary engineer. One of the special difficulties encountered "was the building of a sewage tank 50 feet in diameter, with walls of brick 2 feet in thickness, exteriorly diminishing to 3 feet at the center, the inferior reference of which was 20 feet below low water. As described in the report referred to, this was accomplished very successfully by excavating a large area to the reference of ground-water, some 5 or 6 feet below

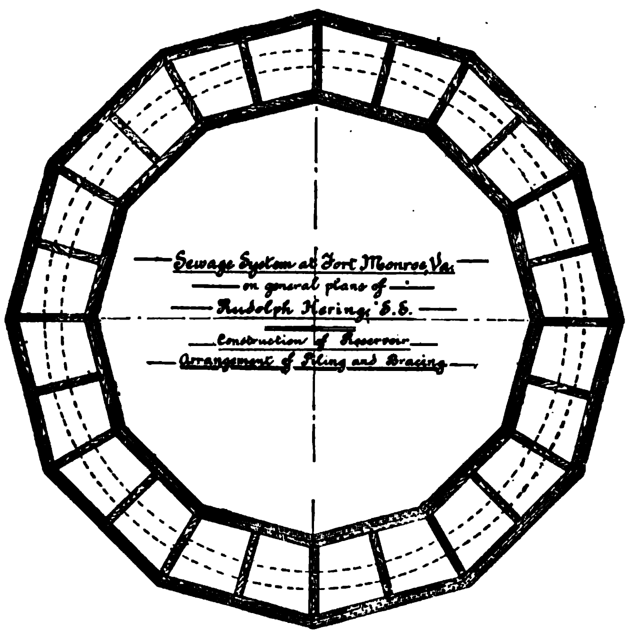


FIG. 87.—RESERVOIR COFFER-DAM. FORT MONROE, VA.

the surface, and then driving by the pile-driver and water-jet combined, two concentric twelve-sided polygons of Wakefield sheet-piling 28 feet in length, 30 and 22 feet from the center, about the circumference of the shallow excavation. (Fig. 87.) The material, consisting of fine water-soaked sand, with a small admixture of clayey matter and fine gravel, was then excavated between the polygons to a reference of 20 feet, transverse shoring braces bearing upon stout stringers being put in at intervals as the work proceeded. The material did not vary much in its general nature, but a number of old piles were taken up, some of which did considerable injury

to the sheet-piling when driven, as shown in the subsequent excavation. The water was controlled by a powerful steam-pump having its point of suction fixed, the water being permitted to flow toward it throughout the circumference. It was noticed that ground-water came through the sheeting very freely at first, but that it constantly ceased to flow to any great extent at a height of a few feet above a point of excavation as this continually descended, owing to the rapid drainage of the strata. The interior core, in fact, became quite dry, so that in excavating after the walls were laid, no water was encountered until the bottom of the external concrete ring had been virtually laid bare. Upon attaining the reference—20 feet, the excavation ceased and hand-mixed concrete was deposited directly upon the bottom, as this was considered to be sufficiently firm, the pump being stopped temporarily in order to prevent a flow. The concrete was rammed firmly against the outer sheeting externally and against plank forms with triangular cross-section resting against the inner sheeting internally, until 6 feet in depth had been put in place. The portion of the ring at the pump suction was filled rapidly with concrete in bags. The 2-feet brick wall was then carried up from the axial line of the concrete ring, the space between the wall and the outer sheeting filled with sand, except about 6 inches at the base of the wall, which was of concrete. The braces were removed as successively attained, the inner prism of dry sand being held securely by the sheeting and the extreme top struts, which were left in place until the inner core was completely excavated. On the completion of the latter work to reference—20 feet, the water which came in freely from without under the concrete ring at several points was conducted in a peripheral trench to the fixed point of pumping. No water came upward and the middle portions of the bottom became perfectly dry. The inner sheeting was cut off at the base of the ring, boards were placed transversely over the peripheral trench, a duck tarpaulin coated with hot asphalt laid down, and concrete rammed in place until the concave bottom with sump channel had been completed, leaving only the pipe, through which the ground-water had been pumped continually, night and day at about 1000 gallons per minute, penetrating the concrete. In order to fill this pipe, it was cut off above the level of permanent ground-water, and after the water within had attained the level of ground-water in the surrounding area and had become perfectly quiescent, neat cement in paper bags was dropped within, being retained at the bottom by the closed valve; the bags were readily broken up by a long pole thrust down the pipe. The latter was then cut off at the level of the bottom

and a coating of cement plaster applied throughout. The resultant leakage through the bottom did not exceed about a gallon a minute and this will be greatly reduced by the infiltration of sand from beneath."

Further illustrations of the use of sheet-pile coffer-dams will be given; then the operations of dredging, pumping, and concreting will be described at some length.

The piers for a bascule bridge over the Lake Washington Canal in Seattle constructed by the author for the Northern Pacific Railway under W. L. Darling, Chief Engineer of the road, were built from plans prepared by H. E. Stevens, Bridge Engineer of the line.

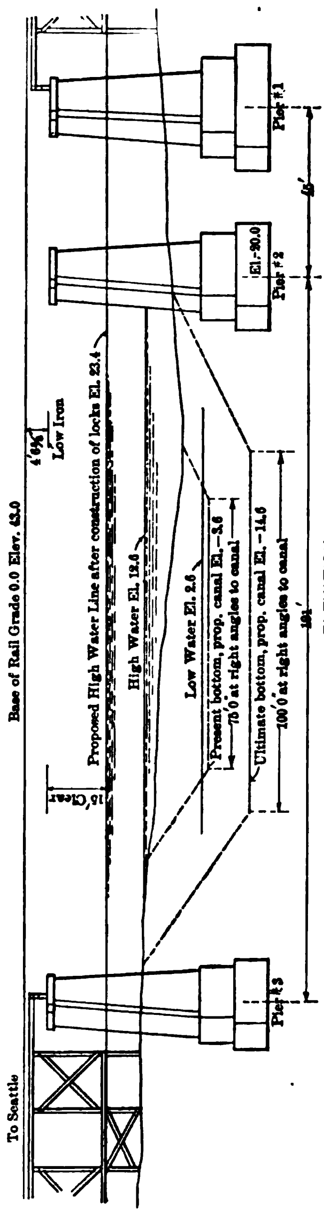


Fig. 88.—Salmon Bay Piers, Northern Pacific Ry.

Piers No. 1 and No. 2 on the north side of the canal are 45 feet centers, while pier No. 3 is 191 feet from the center of pier No. 2 and across the canal, as shown in Fig. 88. The borings showed (Fig. 89) a thick layer of wet, soft brown loam beneath the surface; next a layer of soft blue clay, and underlaid by a bed of fine sand carrying 1 per cent. of fine gravel. The plans indicated the probable use of piling under all three piers. The total weight on the foundation of pier No. 1 was calculated to be 6323 tons, on No. 2 to be 4520 tons and on No. 3 to be 3382 tons; giving a pressure on the sand in case piling was not used and taking into account the bouyancy, of 2.98 tons per square foot for No. 1, of 2.81 tons for No. 2, and 2.84 tons for No. 3. In case piling was used as contemplated, the load per pile considering buoyance would

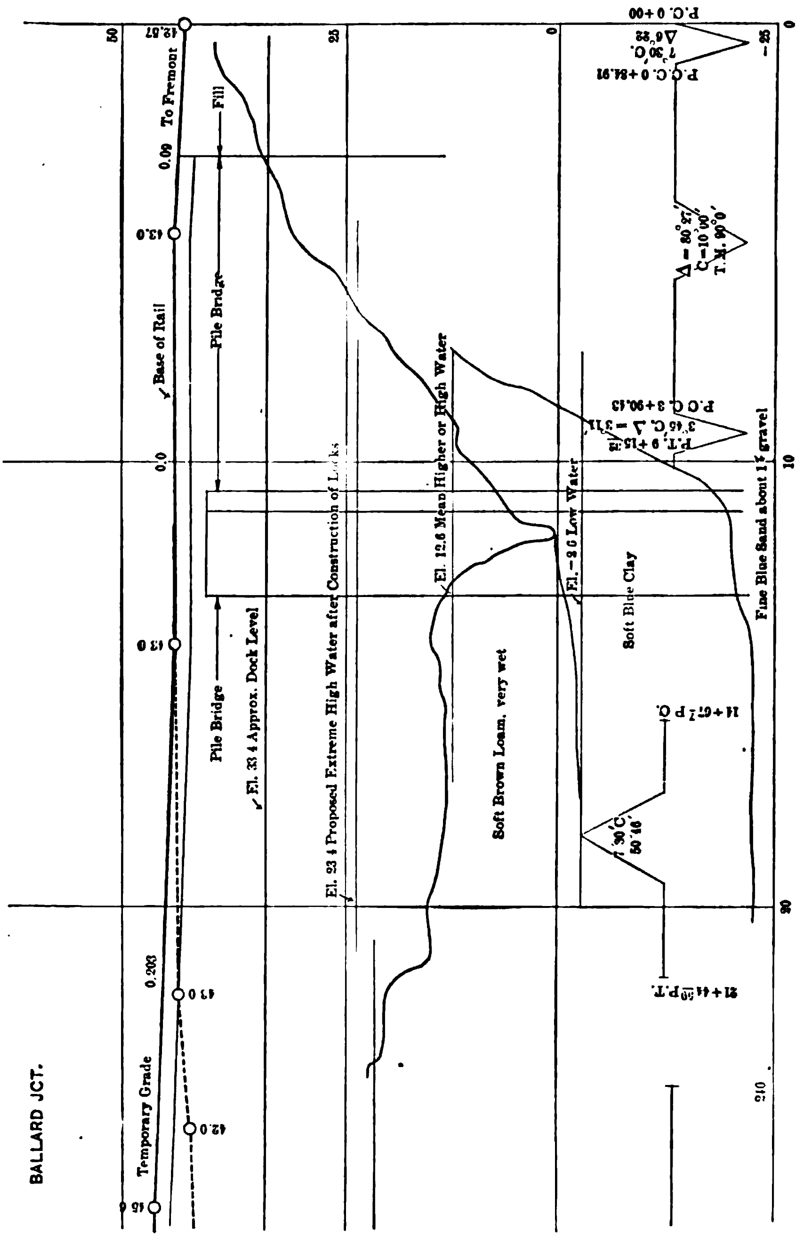


FIG. 89.—SALMON BAY PIERS, N. P. RY. BORINGS.

be for No. 1 pier 21.8 tons, for No. 2 pier 21.0 tons, and for No. 3 pier 21.7 tons. Considering that one ton per square foot of base was carried by the sand, each pile in No. 1 would have to sustain 18.9 tons, No. 2 pier 17.2 tons, and No. 3 pier 19.0 tons.

With the bottoms of the piers at minus 20 and the ground surface near high tide elevation of plus 12.6, and with the soft watertight material to penetrate, the location seemed an ideal one for using sheet-piling for coffer-dams. Accordingly sheet-piles were prepared

FIG. 90.—COFFER-DAM N. P. PIERS.

of 10×12 and 12×12 timber, with dove-tail tongue and groove spiked on (Fig. 91); but upon proceeding to drive them, a hard bottom was struck on pier No. 1 at about elevation −8 to −11, and on pier No. 2 at about −11 to −14. This was found to be so hard, apparently cemented gravel, that it was believed it would stand up while excavating below the bottoms of the sheet-piles, especially if proper lagging or short sheeting was used. Work was continued and all of piers No. 1 and No. 2 were driven, but upon proceeding to excavate pier No. 2, carrying down the bracing as the hole was deepened, the

whole mud flat commenced to slide towards the canal, carrying the tops of Nos. 1 and 2 coffer-dams with it.

Work was then stopped on No. 2 and the decision made to complete No. 1 first to act as a retaining wall to protect No. 2 and upon reaching the bottom of the sheet-piling, it was found impossible to use the lagging on account of the hard material going to pieces when exposed to water. To overcome this difficulty sheeting was driven outside the sheet-piles, of 4×12 plank, 40 feet long, and by this means a depth of -16.5 was reached, where it was decided to found the

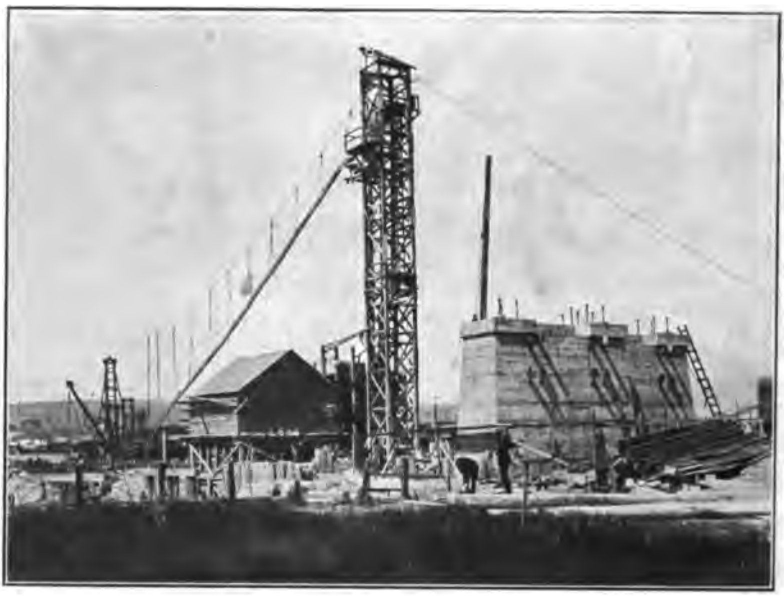


FIG. 90a.—SALMON BAY PIERS, N. P. RY.

pier on the hard bottom without piling. The excavation was all done by a one-yard Owen clamshell bucket, operated on a derrick, from a double $8\frac{1}{2}\times 10$ hoist engine, with swinging drums. This bucket dumped into a hopper at the end of a sluice 24 inches wide and 12 inches deep, which was supplied with water to wash away the excavated material, from a 6-inch direct-connected sand pump. This drew its water from inside the coffer-dam, and with the assistance of a No. 2 Emerson pump kept the dam dry. The steam for these and a $7\times 4\frac{1}{2}\times 10$ jetting pump was supplied by an 80 horse-power vertical boiler.

The material was kept away from the bucket by the sluiceway

described, and some of the material was so slick it would slide in chunks along the sluice, in pieces of a quarter of a yard or more, a distance of about 150 feet to the grade fill back of the bridge.

Upon pumping out No. 2 again and finding that the 4×12 sheeting would have to be driven around it, the decision was reached to complete pier No. 3 next, in order not to disturb the cement house

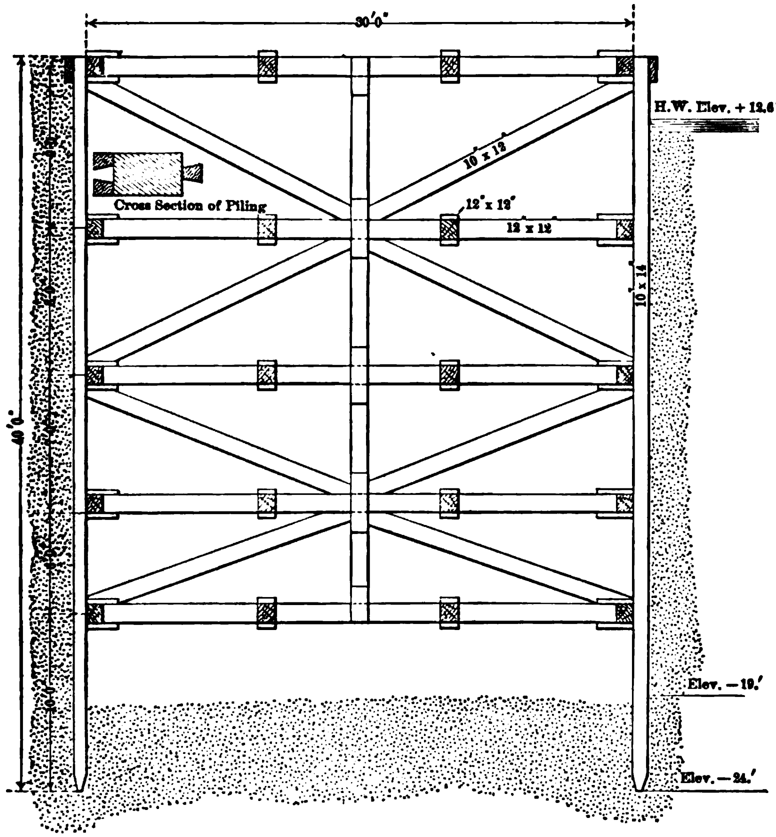


Fig. 91.—Salmon Bay Coffer-dam Pier No. 3.

and spouting tower. (Fig. 90*a*.) The sheet piles for No. 3 of 10×12 timber 40 feet long had been driven, and at only this short distance of less than 200 feet from the other piers no hard stuff was struck, and they reached a penetration below −20, the bottom of the pier. This was excavated in the same manner as described for the other piers, the bracing being placed as the hole was dug out (Fig. 91). Bottom was reached without encountering any hard material, but the material

proved to be full of water, which bubbled up through it and made the sand act much as quicksand, swelling up also, so for quite a time no gain in depth was made, and it was decided to drive piling to carry the pier. This was done by means of leads swung from the derrick. The jet was run down into the bottom as described in the chapter on Jetting Piles, the piles then placed and driven home to a penetration of from 25 to 40 feet. The coffer-dam on this pier kept its shape and was easily kept dry by No. 2 and No. 3 Emerson pumps. After the piles had been driven and cut off, the swelled material from the driving was excavated, but as it kept swelling up, it was finally decided to concrete the base at −19. At extreme high spring tides the water came in over the top of the sheet-piles and work had to be stopped for part of each day for some days.

This is a notable example of the successful use of sheet-piling to an exceptional depth, and had the borings panned out no trouble would have been experienced on any of the piers. Had the borings indicated the real nature of the material, metal sheet-piles might have been employed and probably would have penetrated the hard bottom, and the difficulty have been overcome. The lesson to be learned is that any kind of borings, except core borings, are unreliable and are liable to be very misleading.

Upon the completion of pier No. 3, work was resumed on No. 2. The second line of sheeting of 4×12 planking, 40 feet long, was driven and a penetration of several feet more obtained than was realized with the original sheet-piling.

With the exception of a few small breaks during the excavation, which were easily closed by driving the plank deeper, or by inside lagging, no trouble was had, and when elevation −18 was reached, the bottom was leveled off and the concrete base poured. The sheet-piling was removed from all the piers by boring holes into it and shooting off with dynamite, using two sticks to a pile.

CHAPTER VII

CONSTRUCTION WITH SHEET-PILES (CONTINUED)

VARIOUS combinations of the sheet-piling shown in Fig. 52 may be made, when occasion demands, or modifications may be made that will perhaps render the available material more effective. For example, the form (*g*) may be modified to the form shown in Fig. 92, which has the advantage of a wider lap, and should the piles not draw tight together in driving, no crack will be left open to admit the water. Then the piles of this form will act as guides to the ones being driven, similar to the ordinary tongue-and-groove piling.

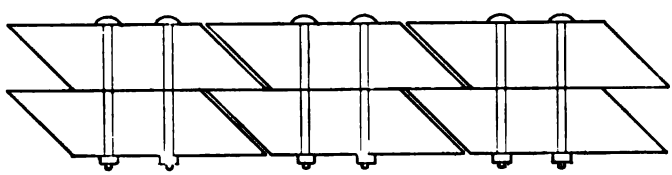


FIG. 92.—COMPOUND SHEET-PILE.

Other combinations and arrangements will readily suggest themselves as necessity may demand.

The use of sheet-piling is often accompanied by a great deal of trouble in securing tightness, and as a matter of precaution, the very best method possible should be adopted in making the piling.

The coffer-dams constructed at Chattanooga for the Walnut Street bridge over the Tennessee River, under Edwin Thacher, consulting engineer, were described in the *Engineering News* of May 16, 1891.

Four piers were founded by this method, but the account of pier No. 2 will fully illustrate the work. The bed-rock, which was level, was covered by cemented sand, gravel, and boulders, of which 320 yards were removed. The coffer-dam was built 18 feet high, or 8 feet above low water, to provide for a future rise. The inside was made large enough to allow of a space of 4 feet all around the base of the pier, and the space between the sheet-piles for a puddle-

chamber was made 9 feet. This was filled to an average of 12 feet with a clay puddle, of which there was 900 yards used. As a protection, there was placed outside the dam about 450 yards of puddle, and a breakwater was built up-stream. About 38,000 feet of timber was used in the dam and breakwater.

After the dam was completed a rise of 30 feet washed out about half the puddle, and one end was crushed by a raft, but the repairs were made without serious trouble. No extra amount of pumping was required on any of this work except pier No. 3, where the seams in the bed-rock required pumps with a capacity of 5000 gallons per minute, and these did not suffice to keep the water down, until the seams were closed by laying sacks of concrete over them and weight-

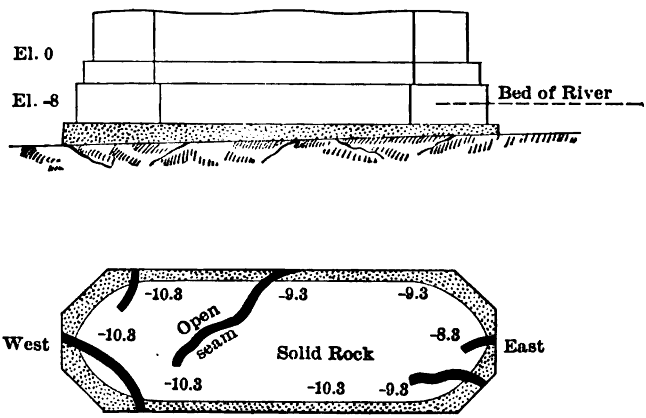


FIG. 93.—CHATTANOOGA BRIDGE, BED-ROCK PIER NO. 3.

ing them down with large stones. The location of these seams is shown in Fig. 93.

The framework and wales for a sheet-pile coffer-dam, used in founding the pier for the Baltimore Street bridge at Cumberland, Md., are shown in Fig. 94, and this was described in the *Engineering News* of July 21, 1892, by H. P. Le Fevre, engineer in charge. The frame was built in place on two canal-boats and after completion was suspended from the old Bollman truss which the new bridge replaced.

The depth of the water was 4 feet, and about 6 feet of very loose quicksand and small round pebbles overlaid the hard bottom.

After the boats were removed, the frame was lowered to its place the sheet-piling driven, and the dam pumped out with a 6-inch pump. The foundation was laid on the hard bottom under the quicksand, after this had been removed.

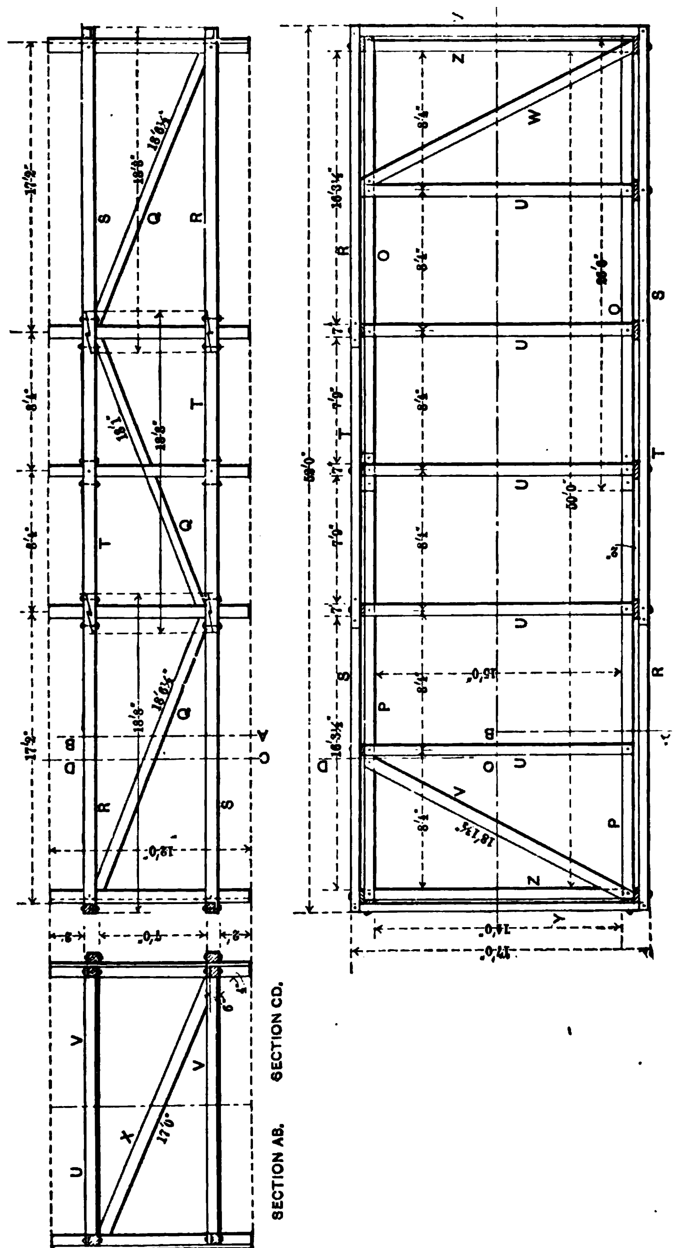


FIG. 94.—FRAMEWORK OF COFFER-DAM, CUMBERLAND, MD.

The grillage was made of two courses of 15×15-inch clear white oak, around which was built a framework, and the open spaces of the grillage were then filled with a concrete made up of one part of Cedar Cliff cement to two parts of sand and four parts of hydraulic limestone broken to pass through a 2-inch ring. Upon this were laid the footing courses of the masonry.

Another ordinary sheet-pile coffer-dam which gave good satisfaction was used at the Sandy Lake dam on the Mississippi River, by Major W. A. Jones, Corps of Engineers, and as the account contains so much of value it will be quoted in full from the 1894 report of the Chief of Engineers.

"The coffer-dam is composed of two rows of round piles, 12 feet from center to center of piles, with the exception of 62 feet of the east end of the upper part, where they were driven 14 feet from center. The piles in each row are $8\frac{1}{2}$ feet from center to center,

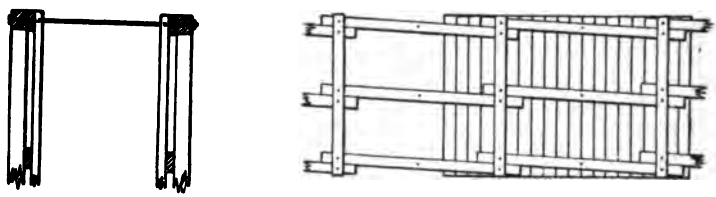


FIG. 95.—SANDY LAKE COFFER-DAM.

cut off at an elevation of 1217 feet above sea-level and capped with 12×12-inch timber. The inside row of sheeting is 4×12-inch, and the outside 6×12-inch plank. The sheeting is cut off at an elevation of 1218 feet above sea-level, or 2 feet below the flowage line. One-inch rods of round iron, $8\frac{1}{2}$ feet apart, pass through the caps to prevent the filling from spreading the two lines of sheeting at the top.

"In May, 1892, when a flood occurred, the outside of the coffer-dam was raised 3 feet by splicing 3-inch planks to the outside row of sheeting and then filling the triangular prism thus formed with earth. The cross-section of Fig. 95 gives an idea of the dam above the bottom, while the longitudinal section shows the framing down to where it rests on the bottom, the frames being joined by the 1-inch lateral rods of iron.

"The total length of the coffer-dam is 829 feet, of which 742 feet is like that shown in cross-section and the other 87 feet like that shown in the longitudinal section.

"The number of round piles driven in the foundation is 1605. The driving was commenced on November 12, 1891, and completed on August 21, 1893.

"The material in the foundation is sand, excepting in the lower right-hand corner, where there is some blue clay overlying the sand. The sand in the foundation is not as compact as it is usually found in the bed of streams. In the south half of the dam, the surface settled from 6 to 4 inches during the driving. As the surface was settling, the driving became harder all the time. In the north half, which embraces the navigable pass, there was some settlement, but it was not as noticeable as in the south half. The surface had probably settled by the jarring of the hammers while the first half was being driven. The penetration of the piles is also greater than it usually is in sand foundations in the bed of streams.

"The piles were all of Norway pine and well seasoned. Two Mundy steam hoisting-engines were used in driving, one a single-cylinder and the other a double-cylinder engine. In operating the hammer a ½-inch manila rope was attached to the pin connecting the lugs of the hammer, then passed over the sheave at the top of the leaders, and next around the drum of the hoisting-engine.

"When the hammer falls, it pulls the rope with it and unwinds it from the drum. This is what is termed driving with a 'slack line.' The blows are more rapid and keep the material around the piles looser than it would be in the case of using nippers. Iron rings of $\frac{5}{8} \times 2\frac{1}{2}$ inch Norway iron were used to protect the head of the pile.

"It is a well-known fact in pile-driving that it is very important to keep the material from settling around the pile, once it has been loosened, until the pile is down; for when the material has settled, or even partially, the penetration is diminished. The greatest load on a bearing pile is about 13½ tons.

"Sheet-piling was driven by a pile-driver, assisted by a jet of water from a steam force-pump. In driving all sheet-piles a cast-iron cap or follower was used which fitted over the head of the pile. On the upper side of the follower there is a wooden block of some seasoned or close-grained wood which receives the blow of the hammer. This device saves the head of the pile from being battered or splintered, and the pile can be driven to a greater depth than it could be without it.

"In first using the jet on a sheet-pile, a groove was made in the inner edge to receive a ½-inch gas-pipe, which was connected to the force-pump by means of a 1½-inch hose. The aperture at the lower

end of the gas-pipe was reduced to a diameter of about $\frac{3}{8}$ inch. The water was thus forced to the bottom of the pile and the sand loosened.

"This worked well until the sheet-pile struck gravel, when the nozzle of the pipe would become battered or filled with gravel. The pressure in the hose would then burst a coupling somewhere. Another source of trouble was the frequent breakages in the connection between the pipe and the hose, on account of the jarring of the hammer. This plan after a while was abandoned and the nozzle of the pipe was thrust by hand under the point of the pile. The piles are driven in the ground from 12 to 14 feet."

The construction of the Main Street bridge at Little Rock, Ark., involved the construction of two coffer-dams, for piers No. 9 and No. 6. This work was done under the direction of Edwin Thacher, consulting engineer, whose original specifications called for pile foundations for these piers, the piles to be driven to bed-rock and cut off 4 feet below water, to receive a grillage of 12×12-inch timbers to receive the masonry. The size of the grillage being 12 and 13 feet wide by 34 feet long and resting on forty-eight and sixty piles respectively, the piles being of good sound oak or pine at least 7 inches in size at the small end and not less than 12 inches at the butt when sawed off.

The coffer-dams were constructed, as can be seen from the view in Fig. 96, by driving guide-piles, to the top of which are drift-bolted square guide-timbers. The sheet-piling of 3-inch tongue-and-groove stuff was driven against the outside of this timber, and the excavation banked up against the outside. They gave excellent satisfaction and caused little trouble, as the water was shallow.

The piers were constructed of Portland cement concrete, the facing of 2 inches thickness being a mortar of one part cement to two parts of sand, while the balance was of concrete of one part cement, three parts sand, and six parts of broken stone.

Where sheet-piles are to be driven on rock bottom or through earth or gravel to rock bottom, they should be driven hard enough to broom up and form a close joint with the rock. This has been accomplished also by driving the piles with a thin edge until they fit the rock bottom, when they are drawn and after cutting them to conform to the contour of the rock, they are redriven, thus forming a tight joint. This method, while very good, is too expensive for general adoption.

The construction of the piers for the Philadelphia and Reading Railroad bridge over the Schuylkill was accomplished by the use of a floating coffer-dam, the foundations being laid upon the bed-rock.

When in position for work the dam is rectangular in shape, 62 feet long and 36 feet wide, outside dimensions, and 16 feet high. Each side consists of timber crib-work 10 feet wide, making the inside

FIG. 96.—COFFER-DAM AND CONCRETE PIER, LITTLE ROCK, ARK.

dimensions 42′×16′. At each corner there is a movable timber extending vertically from the bottom of the crib to some distance above the top. These timbers or spuds are shod with iron on the bottom, and serve to hold the dam in position while the sheet-piling is being driven.

The dam is divided vertically through each short side into two equal parts, which can be floated separately to any desired position and afterwards joined together. Water-tight compartments are also used to hold stone when it is desired to sink the cribs.

When the two sections are united and placed in required position the spuds are dropped and the crib-work is sunk by letting water into the water-tight compartments, and putting in the necessary amount of stone.

Any irregularity in bearing between the bottom rock and the bottom of the crib is then corrected by a diver, who blocks up where required. Close sheet-piling of jointed plank 3 or 4 inches thick is then put on the outside and spiked to the cribs. Puddle, composed of clay and gravel, is then thrown around the bottom outside, and the dam is ready to be pumped out. When the masonry reached the height of the braces they were taken out and the dam was braced against the masonry.

The maximum depth of water encountered at Falls Bridge was 13 feet at ordinary water-level. Several freshets occurred during the progress of the work which did some damage to the dam. At one time, when a dam was ready to be pumped out, a rise in the river moved it down-stream about 30 feet, tearing off the sheet-piling. It was drawn back to place and successfully completed. To make a complete shift of the dam from one pier to the next, with a gang of six men, required about six or eight days, divided as follows: To take the dam apart and reset it, about three days; to sheet-pile, about two days; to puddle, about one day; and pumping out and puddling meanwhile required about one to two days, depending on the amount of the leakage. At each shift, a portion of the plank sheet-piling, perhaps 10 per cent., had to be replaced by new stuff. The pump used was located on a small steamboat, and was run by a steam-engine. The amount of pumping required after the dam was once pumped out varied for the different piers; some dams required little pumping and others a good deal. Only one of the foundations required much leveling off of the river bed, and this one also gave considerable trouble to keep the water out, but the leaks were finally stopped by using gunny bags around them, the bags being drawn into the crevices by the force of the water, thus holding the puddle.

The floating dam was used for three piers in the river channel, the two piers near the shore being put in with ordinary dams. The floating dam is still in good condition and could be used again if needed. The original dam, of which the one used at the Falls Bridge is an enlarged copy, was used for twenty-three or twenty-four settings

The foregoing account is taken from the *Engineering News* of May 24, 1894, the description being by W. B. Riegner, who states also that the cost of the coffer-dam, including one set of sheet-piling, was $3000, while the total cost for five coffer-dams, including the two crib coffer-dams at the sides of the river, was $14,000.

The subject of subaqueous foundations has been very fully treated in a series of lectures by W. R. Kinipple, M. Inst, C. E., before the Royal Engineers' Institute at Chatham, England.

The use of 6-inch pitch-pine close sheeting was made use of by him for a quay wall in the harbor of St. Helier, Jersey. They were driven to rock or as deep as possible with a 2800-pound hammer, and the tops cut off a few feet beneath half-tide level, and clayey material banked up against the outside. The bottom through which the sheet-piles were driven was sand and clay.

The rock was laid bare to a depth of as much as 13 feet below low water and in sections which contained about 900 tons of water to be pumped out; this was done with a 16-inch centrifugal pump in about forty-two minutes.

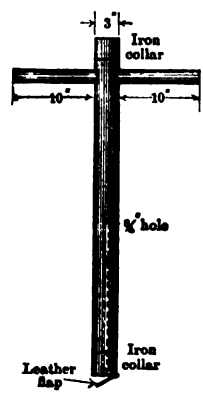


FIG. 97.—STOCK RAMMER.

Several leaks were developed under the piles, but they were promptly stopped by "stock ramming." The stock rammer which is shown in Fig. 97 is 3 inches in diameter, 3½ feet long, and banded top and bottom with iron. A ¾-inch air-hole is bored up from its foot a distance of 20 to 30 inches, and covered on the bottom with a soleleather flap, so that air is let in and suction prevented as it is withdrawn. The sheet-piles have 3½-inch holes bored through their sides, and cylinders of clay are inserted 3"×9" long, similar to the work at Sault Ste. Marie. The stock rammer is inserted and driven by mauls as far as its length will permit, when it is drawn out, and other charges inserted until no more clay can be driven, the hole in the pile being filled with a wooden plug.

The piers for the Putney bridge, over the Thames, were built by the same engineer, with single pile-dams to a great depth, by using 14-inch square piles, with elm-wood tongues, and driving them down through the mud and clay to the stiff clay bottom, so that practically water-tight work was secured.

In the construction of the docks at Victoria, British Columbia, he constructed a coffer-dam 500 feet in length, in a depth of 35 feet

of water, the bottom being of rock and overlaid in places with sand and shells several feet in thickness. At the center the sand and shells overlaid a bed of clay.

Three rows of close 12×12-inch sheet-piling were driven with two puddle-chambers of 7 feet each between. The guide-piles were 15×15 inches and the wales were 12×12 inches.

Where the dam rested on rock at the ends, heavy shoes were used on the piles and concrete deposited around their feet to make the work water-tight. This dam was completed in October, 1879, and remained thoroughly tight until the dock was completed over seven years later.

The arch bridge at Topeka, Kansas, over the Kaw River, which is being constructed on the Melan system, of concrete and steel, by Keepers and Thacher, the designing engineers, is a most interesting piece of work. The coffer-dams were required by the specifications to be water-tight, and to effect this 4×12-inch tongue-and-groove sheet-piling was used. The size of the coffer-dam for pier No. 4 was 18×55 feet in the clear (Fig. 98) and the piling was driven about 16 feet into the sand bottom or 22 feet below low water. The driving was done by a 1600-pound hammer with 36 feet leads, the power being furnished by a 15-H.P. hoisting-engine.

No puddle was used around the outside except to stop leaks, and the dam was kept clear of water with a No. 6 Special Van Wie sand-pump. The capacity of the pump was 1500 gallons per minute of water, and from 60 to 80 yards of sand per hour. It was operated with a 15-H.P. engine. The other piers were handled in a similar manner and with no particular trouble.

The growing scarcity of timber will doubtless lead to the exclusive use of metal at some time in the future, to replace sheet-piling for coffer-dams, but where timber is abundant and reasonable care is exercised in its use, it will continue to be of great service in obtaining foundations by this method.

The following account of the construction and failure of the coffer-dam at Dam No. 48 on the Ohio River is taken from the account by J. C. Oakes, Maj. Corps of Engineers, in Professional Memoirs.

"Of the fifty-four dams proposed for the canalization of the Ohio River all of those constructed up to the present time, except No. 37 below Cincinnati and 41 at Louisville, have been constructed in the upper 300 miles of the river above the mouth of the Big Sandy. One dam, No. 29, is now under construction just above Ashland at mile 320, and contracts have been let and coffer-dam constructed for

locks and dams No. 31 at mile 358¼ and No. 48 at mile 804½, or 6 miles below Henderson, Ky.

"All of the dams thus far built or contracted for, except No. 48, have fairly firm foundations, most of them being on rock and a few on gravel. The material at all lock sites above Louisville is supposed to be of a character that will not be easily eroded by the current, so that up to the present time no special precautions have had to be taken for protection of coffer-dams during construction or for protection of the works against undermining by the current after completion and during operation.

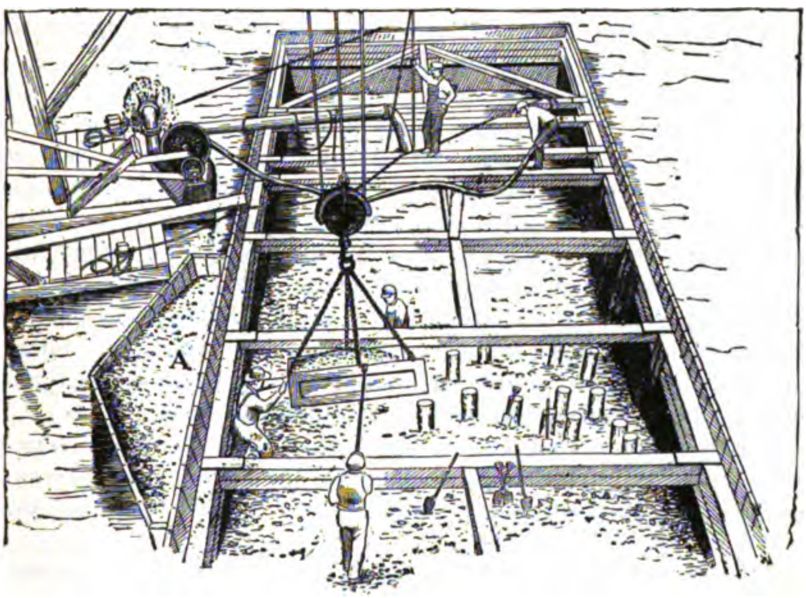


Fig. 98.—Topeka Bridge, Coffer-dam No. 4.
"A" shows puddle to stop leak.

"Below Louisville, however, a totally different type of foundation is encountered. In the thirteen dams to be constructed in the lower 400 miles of the river, rock is found at the sites of only three; elsewhere foundations are fine sand and silt, so fine that the bottom changes with every stage of the river. This fact has caused considerable anxiety, not only with reference to the planning of the works to insure stability after completion, but particularly with reference to the danger to the coffer-dams and erosion of the bed of the river during the period of construction. It has been openly affirmed by some of the contractors who have had experience on the Ohio River

that it would be impossible to construct coffer-dams in the shifting sands of the lower river that would remain during the period of construction, and, second, that if constructed, they could not be made sufficiently impervious against seepage to withstand the ordinary pressure heads, and that they could not be pumped out sufficiently to enable the work to proceed.

"Bids were opened in this office for the construction of No. 48 on September 7, 1911, and it was found that only one bidder had sufficient confidence to offer to do the work. On October 11, 1912. bids were to be opened for the construction of Dam No. 43, but no bids were received. The contract for No. 48 was finally awarded to the only bidder, The Ohio River Contract Company, and during the past season the coffer-dam surrounding the lock, enclosing an area of 20 acres, has been constructed, pumped out, and the round piles under the river wall driven. Work was shut down for the winter on December 31, 1912, and during January, 1913, this work was submerged by one of the worst floods of record at that site, high water reaching an elevation of 371, which is within 2 feet of record high water. No particular damage has been done the coffer and it is expected that it will be in as good condition next season as is usual at other sites after the winter floods. It has therefore been proven that safe coffer-dams can be constructed, maintained, and pumped out without undue trouble at the sites in question as well as in other parts of the river where better foundations exist.*

"It is believed that a short description of the work and type of coffer will be of interest to the members of the Corps of Engineers and also to contractors who might be desirous of bidding on future work in this section of the river.

"At the site selected for dam No. 48, the width between low water lines is 1600 feet; between 20-foot contours, 3100 feet. The lock is to be located on the convex shore of a bend in the river, the radius of which is approximately 4000 feet. Low water is at elevation 325 (Sandy Hook datum), with high water (1884) at 373. The flood of January, 1913, at this site almost equaled that of 1884, reaching an elevation of 371.0. The lock is to be constructed on the Indiana side where the general level of the banks is at elevation 360. On the Kentucky side the bank is somewhat higher, being approximately at 380. The river wall of the lock is to be approximately along the line of low water, which places that wall some 700 feet from the contour of the bank corresponding to that of the elevation of the top of

*See pages immediately following for an account of the failure of this coffer-dam.

the walls. The terreplein between gate recesses is to be connected with the river bank by a causeway or dike at a general elevation of the top of the land wall.

"The lock is the standard size adopted for the river, 600 by 110 feet, with lift of 9 feet. The dam is to have a navigable pass 800 feet wide with Chanoine wickets 16 feet 5 inches long; a Chanoine weir 600 feet wide, wickets 11 feet 9 inches long, and a permanent weir 890 feet long, the elevation of whose crest is to be 1 foot below upper pool, or 337. Upper pool is at 338, lower pool 329, top of river wall 341, and top of land wall 343.

"The material found at the site is a fine sand and silt intimately mixed, with an occasional pocket or very fine gravel. In driving a few long piles to a depth of 47 feet below low water some difficulty was experienced and it is supposed that a layer of gravel was encountered, but no borings have been taken to verify this. This material when quite dry or when entirely submerged in still water stands at about the slope of 2 on 3, but any movement of water either through it or over it rapidly flattens such slope to about 1 in 20. For this reason the coffer-dam was kept 150 feet away from the walls.

"Owing to the distance of the lock from the shore and the necessary contraction of the river, it was thought inadvisable to include any of the navigable pass in the first coffer-dam, which was built therefore to inclose only the lock with arms extending to the bank. The area inclosed is about 20 acres. It is feared that this reduction of the cross-section of the stream will cause considerable erosion, and it may happen that the bed of the river will be so greatly changed as to make it necessary to raise the bed before it will be possible to construct the dam as planned.

"The type of coffer (Fig. 99) is that known as the Ohio River box type, built 20 feet above low water and 20 feet thick for the greater part of its length; it consists of two rows of sheet piles 20 feet apart, tied together by steel rods varying in diameter from $\frac{3}{4}$ inch at the top to $1\frac{1}{4}$ inch at the bottom, held apart by separators and held together by wales on the outside 6 by 6 inches at the top, varying to 10 by 10 inches at the bottom. The space between the rows of sheeting was filled with sand removed from within the coffer by a 10-inch suction dredge, the material being excavated from the lock site. The sides of the coffer were carried into the bank, the top being of uniform elevation 20 feet above low water, To increase the stability of the coffer and decrease seepage a line of 7 by 12-inch triple-lap sheet piles, Wakefield, 26 feet long, was driven outside of and around the coffer and bolted to it.

"No special difficulty occurred during the construction of the coffer, and seepage through the coffer was easily controlled by three 15-inch pumps placed on a pump boat resting on piles near the front of and discharging over the coffer. Some difficulty was met in the extension of the upper arm, owing to the fact that the river rose at a critical period and the pile-driving and placing of skeleton of the coffer was carried on in water as deep as 17 feet. As this coffer-dam extends some 700 feet from the bank at this stage there was, of course, a very serious current around the end, and extraordinary means had to be taken to protect the end of the coffer. Round piles with brush between them weighted down by sand bags and rock were used, and after the corner was turned a pile of riprap was placed to protect it permanently against current and ice. The banking along the front of the coffer was protected by several masses of rock forming short spur dikes to prevent a racing current alongside the dike and the whole length of banking was covered with gravel. This gravel was fine, the largest particles being not more than one-half to three-fourths of an inch in diameter, and some of the material was practically nothing but coarse sand.

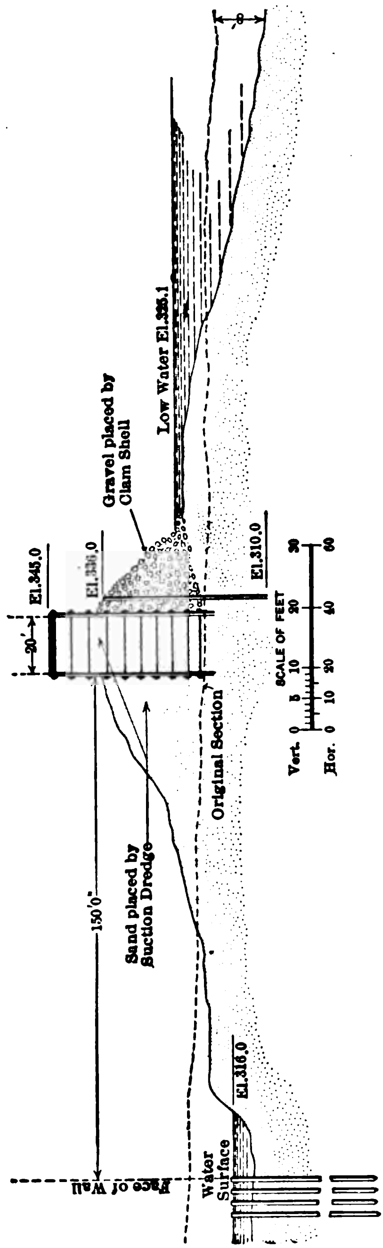


FIG. 99.—TYPICAL SECTION OF COMPLETED COFFER-DAM. DAM NO. 48.

In my opinion, this gravel is much too fine for the purpose intended and the contractor was so notified. He feels, however, confident that it will serve its purpose, and it will be very interesting to see whether his belief is justified.

"The greatest trouble met in the unwatering of the coffer was in protecting the banking against the inside of the coffer from undermining by seepage and drainage water. Wherever there was flowing water the sand was eroded and the banking gradually sloped until it was almost flat. To prevent this, sandbags were used freely, and practically the whole caving surface was covered with the same kind of gravel as that used on the outside. For this protection this gravel was very suitable, and as soon as the sand was covered by a thin layer further erosion did not take place, and the banks remained with a slope of about 2 on 3.

"It is believed that the Wakefield sheet piling driven around the coffer were not used to the best advantage. These piles should have been driven deeper, so as not to overlap the coffer sheeting over 5 feet, instead of about 13 feet as actually driven, and should have been driven close to the coffer. The driving of a great part of the Wakefield piles occurred at medium stages of the river (10 to 17 feet) with the result that the driving was poorly done, and this sheeting is not as tight as it should have been. The contractor was unable to find men who had adequate experience in driving piles in sand with the use of a jet. Both in the case of the sheet-piles and the round piles under the river wall the pile-drivers did not accomplish more than 50 per cent of what they should have accomplished. This was due in part to lack of experience, but also in part to improper pumps, hose, and jets. Up to the end of the season the greatest number of round piles, 30 feet long, driven by one pile-driver in one day was 31. While the jet was used it is very doubtful whether any benefit was obtained, owing to lack of pressure at the nozzle.

"The contractor's attention has been called to these defects, and preparations are being made to provide the pile-drivers with proper jet apparatus, and it is believed that when he begins to drive next season each pile-driver will drive from 80 to 100 piles per day.

"The use of a pump boat with all of the pumps concentrated, the boat resting on piles as the water lowers, has been excellent, and it is believed is very economical and much preferable to other methods commonly used. When the coffer is flooded the boats are simply disconnected from the discharge pipes, their suctions raised, and they are floated out of the coffer through the passway. This furnishes a

very simple and economical manner of removing and placing the pumping apparatus.

"This being the first coffer to be placed on sand foundations in the Ohio River, the contractors have taken very great care to use the best material and to make everything as secure as possible. It is possible that in future coffer-dams some of the measures adopted in this case may be found to be unnecessary and the cost of the coffer-dams materially reduced.

"The following is an extract of a report of the Government inspector in charge of the work, Junior Engineer Edward H. West, in which the work is described in fuller detail: . . .

"The coffer, both inside and out, is heavily banked with sand. A typical cross-section is shown on Fig. 101. Plans were closely followed in construction, the only exceptions being as follows: For a distance of 670 feet from the land end of the up-stream wing and 520 feet from the land end of the down-stream wing, tie-rods were spaced 8 feet on centers instead of 6 feet on centers; in some portions of the coffer, instead of 2 by 12-inch deck joists spaced 24 inches on centers, 2 by 10-inch joists were used, spaced 16½ inches on centers, and in a short portion of the river arm 2 by 10-inch joists 18 inches on centers were used. The sheet piles were driven from a floating pile-driver, and at times the river was too high to permit the piles to be driven to the proper depth; consequently, such piles do not have quite the penetration intended, the maximum difference between actual and intended penetration being about 3 feet; at the up-stream outer corner the length of a number of sheet-piles was increased to 32 feet for additional safety at the point of supposed greatest weakness; at this corner about fifty round piles were driven as shown on Fig. 101; the area inclosed within them was filled with brush weighted with sandbags. Afterward the entire corner was protected with derrick-stone piled as high as the top of the coffer. The lower outer corner was also protected by derrick stone and several stone dikes were built along the river side.

"On June 20, 1912, the line of sheet-piles was commenced, beginning at a point 250 feet from the land end of the upper wing, and piles were driven on fifty-six days, an average of 40 piles per day. On a number of these days, however, two pile-drivers worked, the average number of piles per day for one driver being not more than 30. These sheet-piles were driven rather carelessly, making alignment poor, and leaving the joints not very tight. Better progress could have been made in driving them, but as it was only necessary to keep ahead of the coffer skeleton no great effort was made to push

the pile-driving. After the driving of sheet-piles had been begun, trenches about 2 feet deep and 20 feet apart were dug parallel to the sheet-piles. In these a skeleton was erected consisting of wales and those pieces of sheeting through which the tie-rods passed, the sheeting being driven about 2 feet into the sand. All wales were scarfed for 2 feet at both ends and holes bored for tie-rods through the center of the scarf. Where the spacing of tie-rods was 8 feet 18-foot timbers were used for wales, and where the spacing was 6 feet 20-foot wales were used, thus allowing a lap of 2 feet at each end. At each tie-rod a temporary separator perpendicular to the wales and 20 feet long was placed and the nuts on the rod tightened. The remainder of the sheeting was then driven and after the proper cut-off

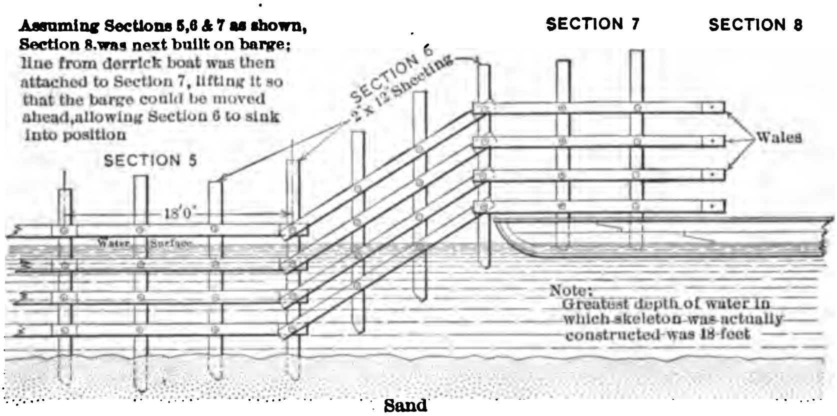


FIG. 100.—METHOD OF EXTENDING 20-FOOT COFFER-DAM. LOCK AND DAM NO. 48.

elevation had been marked the ribbing strips were spiked on and the sheeting cut off to grade. Cracks between adjacent pieces of sheeting were closed by 1×3-inch battens nailed on the inside. When the coffer had been extended into water so deep as to prevent further work from land the skeleton was bolted together on a small barge, one section at a time, and as each section was completed it was lifted by a derrick boat until the barge could be moved forward, when it was lowered into the water (Fig. 100). Sheeting was then driven by men standing on the wales. Throughout the construction two gangs of carpenters worked; at times each gang extended both skeleton and sheeting, but better progress was made when one gang worked on skeleton only and the other followed up with sheeting. Each gang contained at different times from ten to twenty men; it was found that not more than twenty men could work together to

advantage. In general, it may be said that the rate of progress depended upon the rapidity with which the skeleton was advanced, no difficulty being experienced in keeping close up with the other work. As a matter of fact, sheeting was frequently delayed in order that the skeleton might be extended. During the season the skeleton was actually extended on about seventy-one days, an average of 38 feet per day; this value corresponds to two wale lengths, either 36 or 40 feet, depending on the length of the wales being used; it was noted in the field that two wale lengths constituted an average day's work. Sheeting was actually driven on fifty-three days, an average of 52 feet per day (double row) from a minimum of 10 feet to a maximum of 103 feet, but on a great many days on which sheeting was driven much other work was done by the same gang and the above values are not reliable for progress estimates. It is believed that one gang can extend a double row of sheeting 100 feet per day. The extension of skeleton may be assumed to be work accomplished by a single shift, since night work on this portion of the coffer is negligible; sheeting, however, was driven quite satisfactorily at night. After the sheeting had been cut off to proper elevation, the deck joists were laid and spiked to the ribbing strips. This having been done, the discharge pipe of the 10-inch suction dredge was moved into position and the sheeted portion of the coffer filled with sand taken, if possible, from the "pay excavation" for the lock walls. Bulkheads across the coffer were built at convenient intervals as the construction advanced. As the coffer filled, the separators were removed and afterward used again. At first, a long trough built on top of the deck joists of rough lumber, on a grade of 1 per cent., was used for distributing the sand, but it proved to be very inefficient and was discarded; subsequently the desired distribution was accomplished by moving the end of the discharge pipe. Short time tests of the suction dredge showed that it could place 100 cubic yards of fill per hour, through 300 feet of discharge pipe, maximum lift 20 feet. It handled a total of about 100,000 cubic yards of material. The coffer contains only about 40,500 cubic yards; the remainder was used for banking. Of the total excavation made, about 50 per cent. was within the specified limits of "pay excavation." As shown on Fig. 100, a flooding sluice was built in the lower wing and a passway for removing boats, etc., was also left in the lower wing. Discharge sluices and coal tracks were constructed in the river side near the crest line of the dam. A layer of fine gravel was spread over the whole river side of the coffer-banking to prevent scour as much as possible. It is believed

that the one flooding sluice will be all that is necessary, unless very extraordinary conditions arise; in flooding the coffer when the river is near its top and too great a volume of water entering would endanger the permanent work, it is intended to place blocks behind alternate needles, as is often done in regulating pools above needle dams. In the event that this scheme should fail to pass a sufficient quantity of water to fill the coffer before it is topped by a rise, it is intended to use the passway as an emergency opening after constructing a temporary sluiceway.

"The tongue-pieces for sheet-piles were dressed in the contractor's planing mill, as was all material requiring finish, such as sheeting for sluiceways, needles for closing sluices and passway, etc. Two pile-drivers were used during the season, both mounted on decked barges, with cabins. One barge was $20 \times 50 \times 4$ feet (No. 30), drawing 25 inches, and the other (No. 29) $22 \times 60 \times 5$ feet, drawing 25 inches. On No. 30 was one 40-horse-power locomotive-firebox portable boiler, made by the Brownell Company, Dayton, Ohio; one two-drum hoisting engine with four winch-heads, made by the American Hoist and Derrick Company, cylinder diameter 7 inches, length of stroke 10 inches, rated horse-power 20; the hammer was a No. 2 steam-hammer, made by the Vulcan Iron Works, Chicago, weight of moving parts 3000 pounds, gross weight 6500 pounds, strokes per minute 60; the make or size of the pump is not known, but its indicated pressure varied from 80 to 110 pounds; the pressure at the jet is unknown, but pressure loss between pump and jet was considerable; the jet was a 2-inch pipe reduced to $1\frac{1}{2}$ inches at the nozzle. On No. 29 was one 60-horse-power locomotive-firebox portable boiler, made by the Brownell Company; one nondescript engine on Lidgerwood base with two winch heads, cylinder diameter $6\frac{1}{2}$ inches; length of stroke, 8 inches; horse-power, 12. The hammer was a No. 3 steam hammer made by the Vulcan Iron Works; weight of moving parts, 1800 pounds; gross weight, 3800 pounds; number of strokes per minute, 60; a McGowan pump supplied water to the jet at about 90 pounds pressure, as indicated at pump, but it is believed that the greater part of this was lost before reaching the nozzle; the jet was a 2-inch pipe, reduced at nozzle to $1\frac{1}{2}$ inches. At different times, three derrick boats were used; two of these were identical, having been recently built for this contract; the third, No. 31, was an old boat, quite inefficient and nearly worn out. Nos. 33 and 34 were built on hulls $34 \times 66 \times 4$ feet 9 inches, drawing about 20 inches; the cabins are 28×28 feet. The one boiler on each boat is a locomotive-firebox portable boiler, 40-horse-power, made by the

Houston-Stanwood & Gamble Company. of Cincinnati; each engine a 50-horse-power three-drum tandem engine for derrick boats, built by the Lidgerwood Manufacturing Company, cylinder diameter, 10 inches; length of stroke, 12 inches; drums, 16×24 inches; the swinging engine is a 6-horse-power Lidgerwood, cylinder diameter 5 inches, length of stroke 6 inches. The fittings were made by the American Hoist and Derrick Company. The boom is 65 feet long and the mast 36 feet. Each boat has three spuds. All excavation, banking, and filling was done by a suction dredge; hull, 22×100×5 feet 6 inches, drawing 24 inches, with one spud at the suction end. The two boilers are each 60-horse-power, built by the Houston-Stanwood & Gamble Company; one engine, 100-horse-power, built by the same company; cylinder diameter, 15 inches; length of stroke, 20 inches. The pump is a 10-inch centrifugal sand-pump, belt driven, made by the Morris Machine Works, Baldwinsville. N. Y., with 10-inch suction and 12-inch discharge pipes. For pumping out the coffer two-pump boats were available; one, built especially for the purpose, contains, on a hull 36×106×5 feet 6 inches, three 100-horse-power boilers, made by the Houston-Stanwood & Gamble Company; three 100-horse-power engines built by the same company; cylinder diameter, 15 inches; length of stroke, 20 inches; and three 15-inch centrifugal pumps, belt driven, made by the Morris Machine Works, and having 18-inch suction and 15-inch discharge pipes. The boilers consumed 330 bushels of coal per twenty-four hours during a run of eighteen days, and each pump, discharging 26 feet above water surface, handled 8000 gallons per minute. The old pump boat has never been used on this contract; it contains two Brownell boilers, 60-horse-power each; one engine, made by Charles Barnes & Company, Cincinnati, cylinder diameter, 15 inches; length of stroke, 18 inches; one engine, built by the Nagle Engine an Boiler Works, Erie, Pa., cylinder diameter, 12 inches; length of stroke, 16 inches. Both centrifugal pumps were built by the Morris Machine Works, Baldwinsville, N. Y.; one is a 15-inch pump and the other an 8-inch sand pump, both belt-driven. The floating machine shop contains a lathe made by the Hamilton Machine Tool Co., Hamilton, Ohio; a pipe-threading machine made by the Merrell Manufacturing Company, Toledo, Ohio; a pipe-threading machine and a drill-press made by Davis & Egan, Cincinnati, Ohio; and a hand-made forge.

"The coffer-dam was closed November 12, 1912, and a typical cross-section of river side is shown on Fig. 101. Inside were left both pump boats; the old derrick boat for use as a clam-shell dredge;

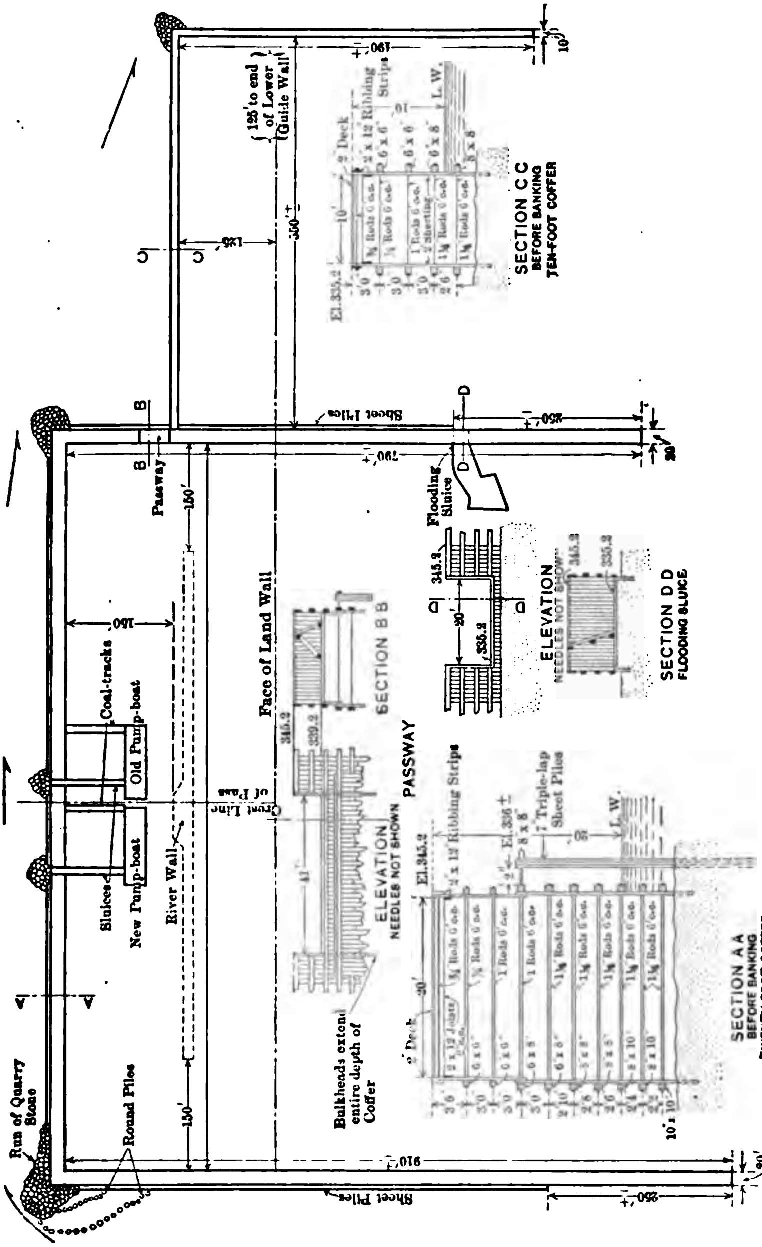


FIG. 101.—COFFER-DAM, OHIO RIVER. LOCK AND DAM NO. 48.

and both pile-drivers. A temporary sluiceway was constructed, discharging through the passway, and a coal track was built to supply fuel to the new pump boat *Uncle Joe*, which was temporarily located over the hole excavated for the lower gate track and recess. It was intended to pump down low enough to enable the pile-drivers to drive 126 piles for supporting the pump boats in their permanent locations. Pumping was begun on Friday, November 15, 1912, at 2 P.M. and was continued until Saturday, November 16, 1912, at 3:10 P.M., when the mouth of the discharge sluice was covered by the rising river and it became necessary to stop the pumps. On Thursday, November 21, the river had fallen sufficiently to begin pumping and the pumps were started at 9:15 A.M. and ran continuously until the piles for pump-boat foundations were driven and capped. Gages had been set inside and outside the coffer and an inspector was kept on the pump boat continuously during the first pumping. The slope of the discharge water was measured in the sluiceway by a series of small gages set before pumping commenced, and the actual discharge while the pumps were making 350 revolutions per minute was computed by Kutter's formula, using a coefficient of rugosity of .009, and found to be 24,000 gallons per minute for three pumps. An attempt was made to obtain more or less accurate estimates of the seepage through the coffer-dam at different heads. Since the pump-boat was floating the vertical distance from water surface to center line of discharge pipes did not vary and the discharge was assumed as constant at 480,000 gallons per pump-hour as measured. The inside gage was read as closely as was possible (to hundredths of a foot) and recorded hourly and a careful record kept of the actual number of pump-hours run. The outside gage was read at 8 A.M., 12 M., and 4 P.M., and the river stage at other hours could readily be estimated. From the readings of these two gages the head on the coffer at any time could be computed. A careful survey had been made of the interior of the coffer by soundings on October 31; from the data obtained by this survey a contour map was plotted on a scale of 1 inch equal 40 feet, contour interval 1 foot, and areas were measured by planimeter at each foot of elevation through the probable range of water surface during pumping. The area of water surface at each hour was obtained by interpolation and the amount by which the water content of the coffer was reduced was computed by average end areas; the seepage during each hour was taken as the difference between the amount of water handled by the pumps and the amount by which the coffer content was reduced. For example:

Date: November 22, 1912.

Time P.M.	Outside Gage.	Inside Gage.	Head.	Pump-hours run.	Water actually pumped.	Content Reduced.	Seepage.	Seepage. per minute.
7	332.53	319.79	12.7					
				2.00	960,000	144,610	815,390	13,589
8	332.50	319.68	12.8					
				2.00	960,000	259,870	700,130	11,669*
9	332.48	319.48	13.0					
				2.00	960,000	230,510	729,490	12,158†
10	332.45	319.30	13.2					
				2.00	960,000	252,380	707,620	11,794
11	332.43	319.10	13.3					

* Average Head = 13.0.
† Seepage per Minute = 12,052.

"It was impossible to read the inside gage closely enough to make the seepage estimates for each hour agree perfectly, and so in plotting the curve shown (on bottom, Fig. 102) the points plotted are average values for several successive hours. While conditions were such that an absolutely accurate estimate of seepage is impossible, the values as given are believed to be close enough for all practical purposes. It would seem that the seepage should vary as the square root of the head, following the well-known laws for flow through pipes and orifices, the passage of the water through sand amounting to flow through an infinite number of minute pipes, but the values actually obtained as outlined above indicate a straight-line variation, that is, directly as the head (top of Fig. 102).

"It will be noticed that from a head of 13 feet to 15 feet the seepage appears to rapidly decrease and from 15 feet to 18 feet it appears to increase more rapidly than normal. This phenomenon occurred during the night when no attempt was being made to lower the water surface rapidly. One pump had been cut out and the other two speeded up so that they were pumping more than the quantity of water measured at normal speed, but how much more is not known; consequently, the seepage appears to decrease, but in reality such is not the case. After the piles for the pump-boat foundations had been driven and capped the coffer was allowed to fill, the boats were floated into position and the *Uncle Joe* pumped down, allowing them to settle on their foundations. Pumping was continued until the water surface inside reached an elevation of 310.7, when the coffer was accepted as satisfactory; it was then allowed to fill up to 316.0, and driving of round piles for river wall foundation was begun from floating drivers.

"Seepage was apparently uniform around the entire coffer perimeter, since a single large leak would have resulted in great damage and the wrecking of a portion of the coffer-dam. Naturally, this seepage water collected into a relatively small number of rather

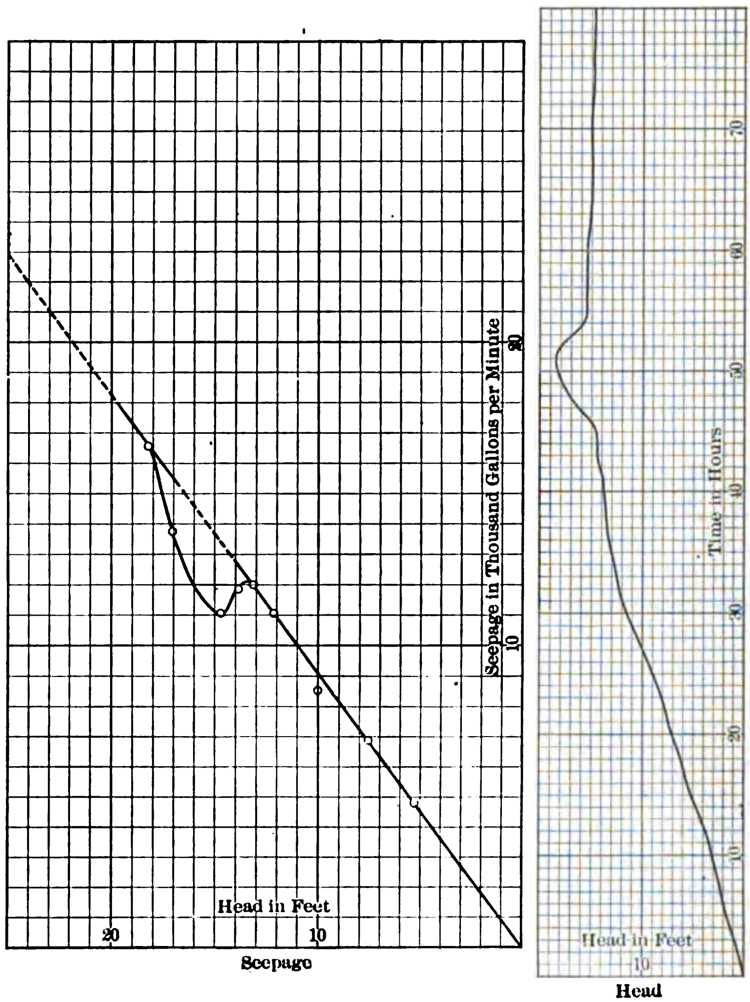


FIG. 102.—SEEPAGE THROUGH 20-FOOT LOCK COFFER. DAM NO. 48.

large streams, moving toward the pumps, each of which carried a large quantity of sand, which was deposited in the still water where the pile-drivers were floating. Taking care of the seepage was a simple matter, but serious difficulty was encountered in preventing the

excavation from filling with this sand. The clam-shell dredge proved to be an inefficient and costly method of solving the problem.

"On several occasions pile-driving was suspended until sand thus washed in could be removed. It was thought that by building dams of sand bags across those streams the movement of sand could be stopped. This was done and proved to be very advantageous, but when water began to flow over the dams sand continued to move into the excavation, though in diminished quantities. A further diminution in the sand movement was effected by spreading a 6-inch layer of gravel over the greater part of the river side of the interior banking. In spite of all efforts, however, large quantities of sand continued to move into the 'hole' and at the time work was suspended for the winter the problem of driving piles without constant excavating was still unsolved. . . .

"At about 7 A.M. Monday, July 21, 1913, there occurred a failure of the main coffer-dam at lock and dam No. 48, Ohio River. This coffer was built 20 feet above low water and at the time of failure the river stage was 12.4 feet above low water, or elevation 337.5 (Sandy Hook datum). The water surface within the coffer was at elevation 316.5, the head being 21 feet.

"The failure occurred at the passway in the lower arm. This passway was left to enable the contractor to pass floating plant in and out of the coffer at stages from 18 to 20 feet, and, as originally constructed, was 41 feet wide with bottom 6 feet below top of coffer, the area being closed by needles, as shown on the illustration.

"After work had closed for the winter and the Government inspector withdrawn, this passway was torn out to remove some floating plant, and without the knowledge of this office was rebuilt with the top 2 feet lower than before. The top of the coffer, therefore, at this passway was only 12 feet above low water, needles being used to close the opening when required. On the outside of the coffer was a line of Wakefield sheet piles 26 feet long, with tops of piles from 10 to 12 feet above low water. These sheet-piles extended about 10 feet below the bottom of the coffer. Sand was banked against the coffer to the height of the tops of the sheet piles.

"In order to hold the banking against the coffer on the inside, the contractors had driven a line of sheet-piles 60 or 65 feet away from the coffer with tops of piles at about low-water elevation and fill had been placed sloping from the tops of these piles to the coffer at the elevation of the floor of the passway. This fill had been covered by gravel to prevent wash by seepage.

"A small quantity of water, probably 1 cubic foot per second, has always collected behind the passway and flowed as a little stream into the excavated area, but as there were may of these little streams, some of them greater than this one, no fear of failure because of this seepage had been felt.

"The Government inspectors lived on a quarterboat moored just below this passway and crossed the coffer constantly at this point, and no increase in the amount of seepage nor any movement of sand had been noticed.

"The excavation was kept clear of water by three 15-inch centrifugal pumps on a pump boat, the latter resting on piles. The engineer of the pump boat noticed nothing unusual until 5:30 A.M. July 21, when he found the pumps were not holding the water surface at 315.5. He then changed his governors, increasing the speed of the engines and finally cut out the governors entirely, allowing the engines to run full speed. In spite of this, the water surface inside rose to 316.5 at 6:30 A.M. Shortly after this the men coming to work noticed a large leak near the passway; the alarm was given, and the pumps stopped at 6:45 A.M. At 7:01 A.M. there was evidently a blow-out, as the water was seen to suck down outside of the sheet-piles; the sheet-piles were lifted out, then the coffer lifted, and the break was completed. Before the inclosure was filled, about 250 feet of coffer had been washed away.

"Through the gap thus created there were drawn four loaded coal barges, a barge of lumber, and one of round piles. The coal barges were rolled over and over, and were a total wreck, the pile barge was broken up, and the lumber barge was injured, but can be recovered and repaired. The pump-boat was thrown off its pile foundation and somewhat injured and four pile-drivers were submerged and probably injured to some extent, the amount of injury being unknown.

"The contractor was about ready to place concrete for the river wall. The excavation was completed, all round piles and most of the sheet-piles were driven and tracks on piles for derricks, cars, etc., were completed. A large amount of sand has been carried into the excavation, coal has been dumped about the heads of the piles and undoubtedly some of the tracks have been injured. It is estimated that the immediate money loss to the contractor will amount to between $10,000 and $15,000, but the accident may cause a much greater loss due to the delay, which is estimated at a month or six weeks, which may prevent the completion of the work inside the coffer this season and make necessary the unwatering again next year.

It is hoped, however, that further investigation when the river falls will show less damage than is anticipated.

" Several causes of the failure can be suggested, and it is probable that they all had a bearing; their relative importance, however, can only be guessed at. These probable causes are: (*a*) The weight of the passway coffer was probably about 1000 pounds per square foot less than it would have been if it had been built to full height; (*b*) in rebuilding the passway, good connection between the old and new sheet-piles and the sheeting of the coffer may not have been obtained; (*c*) the material used for filling the passway coffer when rebuilt may have contained a large proportion of silt as the silt deposit in the coffer during the winter was very heavy; (*d*) the seepage probably gradually increased with the increasing head until, during the night, the little stream began to carry out sand from beneath the coffer and it also probably cut down into the inside banking until the limiting plane of saturation was reached when the blow-out occurred.

" It is certain that there was a considerable increase of flow during the night from this and probably other seepage streams, and that at 6:30 A.M. there was a bad leak under the passway, and when the blowout took place the whole mass became suddenly fluid and lifted the sheet piles and the coffer."

CHAPTER VIII.

METAL CONSTRUCTION.*

THIN steel shells have been used extensively for foundation work, but in the majority of cases they have been retained as essential features of the permanent construction.

This is more particularly the case in locations where stone is scarce or expensive and it becomes necessary to substitute some other material for foundations. Tubular steel piers are constructed of two tubes, ranging from 24 inches to several feet in diameter, or, in the case of pivot piers, from 15 feet, with a single tube for a pier, to 30 feet or more.

In a number of instances the steel shells for ordinary piers have been made oblong, in the general form of a stone pier, and braced internally to hold them in shape during sinking, after which they are filled with concrete.

The metal shells for the Hawkesbury Bridge in Australia were of this character, 20 feet wide, 48 feet long, and with rounded ends. Each one was provided with three dredging-wells, each 8 feet in diameter, through which the dredges shown in the view (Fig. 103) were operated. While these piers were not used as coffer-dams, they were made water-tight by boiler-riveting, so that by pumping water in and out the displacement could be kept constant, and in this way control the pier in an average tide of 5 feet. These piers were sunk by dredging out the material from the inside, to the great depth of from 135 feet 8 inches to 197 feet below the pier tops, or a distance of 155 feet below low water.

Both inclined and vertical cutting edges were used, with the result that the inclined ones were of frequent trouble and the vertical ones none whatever.

* Metal caissons have been used more frequently in the United States than have metal coffer-dams, the reasons being the cheapness of timber and its more easy application.

In England metal coffer-dams are more frequently used. The example given in this chapter—the Forth Bridge coffer-dams—might have been supplemented by reference to those used on the Clarence Bridge at Cardiff, the construction used being illustrated and described in *Engineering*, and is especially notable for the design of the bracing.

"If it is essential to increase the bearing surface at the bottom of the caisson to an area which is not required in the upper portion, this end can be secured by a vertical cutting-shoe of considerable height, with a step or steps into the smaller diameter. This is quite as efficient to secure the end in view as a long incline on the cutting-shoe, and has decided advantages. In the denser material the vertical sides leave the ground undisturbed for some height close to the skin of the caisson, and a vertical guide is secured which is entirely

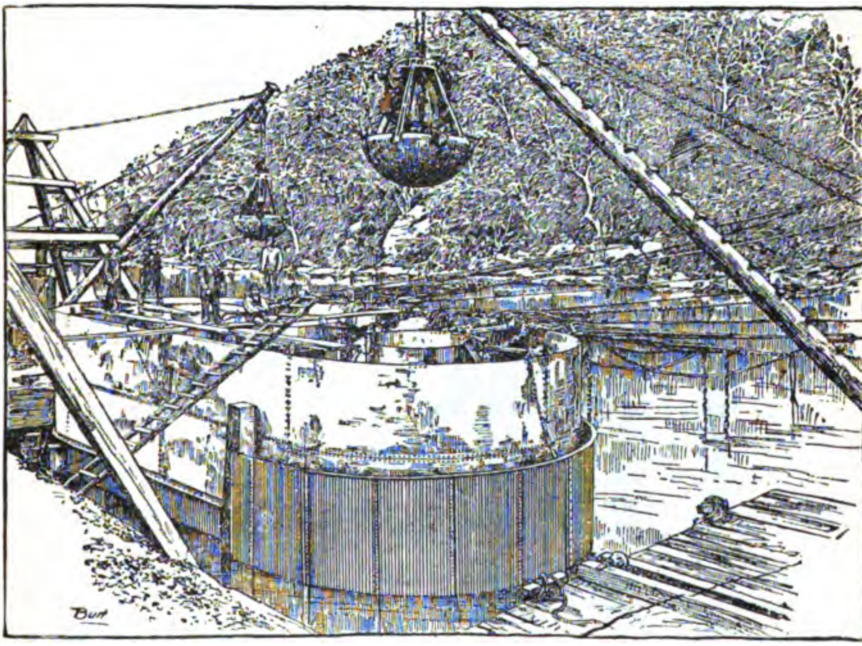


FIG. 103—HAWKESBURY BRIDGE.

Caisson No. 6 in Process of Sinking, Showing Excavator and Shore Chains for Maintaining Vertical Position.

wanting in the case of an inclined shoe. This guide is valuable in cases where the soil may differ in density under the shoe, and particularly so if the excavation has been carried too far below the bottom of the shoe. With an inclined shoe and a slip of soil into the dredging well from one side more than another, experience in deep dredging has shown that there is a decidedly greater tendency to a horizontal movement than with a vertical shoe. The former has a flare to direct this sidewise motion in the first place, and nothing

but a certain amount of disturbed material above the shoe to resist this tendency."

The above account is from the *Engineering News* of January 5, 1889, the work having been done under the direction of J. F. Anderson, of the firm of Anderson & Barr. The shells were filled with concrete up to low water, and masonry built from low water up to the top of the piers.

Such work may be made water-tight by riveting according to ordinary boiler-maker rules, or if extra thick plates are used this can be exceeded and the rivets spaced some farther apart. The joints

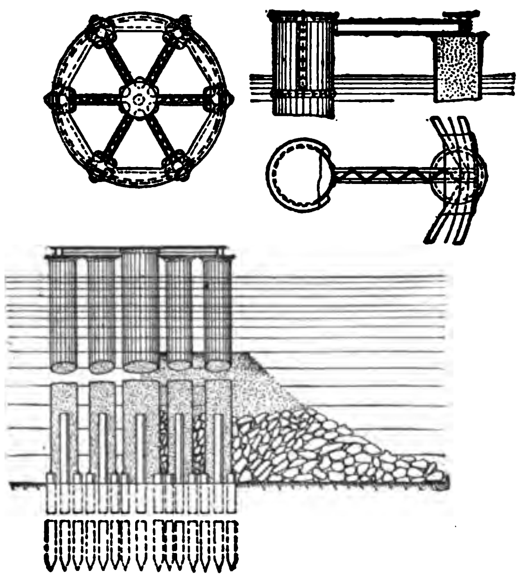


FIG. 104.—GROUP OF CYLINDERS FOR PIVOT PIERS.

may be made with ordinary laps and calked, or a very much better appearance may be obtained by the use of butt joints, and if desirable to avoid calking, then a calking strip may be used to make the joints tight. This is merely a cloth or canvas strip, thoroughly saturated with paint paste, and is laid between the metal surfaces, and the riveting draws the plates upon it and a tight joint will result. The shells will be filled with concrete as soon as the piers are in place and the foundation prepared, so that only a temporary use is required of the strip.

When metal cylinders are used simply as casings for concrete they need not be made water-tight, as they can be dredged out and have

the concrete deposited through the water. The metal should never be less than $\frac{1}{4}$ inch in thickness, and on first-class work $\frac{5}{16}$ to $\frac{1}{2}$ inch is preferable. Railroad work of this character is usually constructed of $\frac{3}{8}$-inch metal for ordinary depths.

The pivot pier of the bridge over the Little Bras d'Or River in Cape Breton was constructed of seven metal cylinders braced together. The center tube was 4 feet in diameter, while the six outside cylinders were 3 feet in diameter. (Fig. 104.) The center pivot, about which the span revolves, rests on the center tube, while the track is supported by the other tubes, but resting directly on rolled beams covered with $\frac{3}{8}$-inch plate.

The tubes rest on a clump of piles, cut off at the bed of the stream, with one pile extending up into the center of each tube about 6 feet, around which the concrete was deposited, thus preventing displacement. Concrete and stone were placed on the outside up to 15 feet, as a protection.

This work was described by Martin Murphy in Trans. Am. Soc. C. E., Vol. 29, who also describes a pier for the Victoria Bridge, over Bear River, constructed with two tubes, resting on piles cut off at the bed of the stream, but having four piles inside each tube. (Fig. 105.) Around the outside are timber, concrete, and broken stone as a protection. The saw used for cutting off the piles under the water was very much simpler than the one shown in Fig. 49, and is illustrated in Fig. 106.

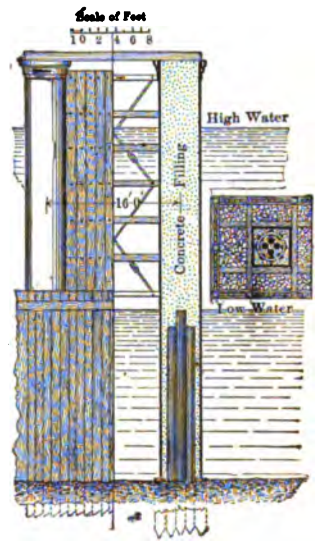


FIG. 105.—PIER OF TWO CYLINDERS, VICTORIA BRIDGE.

Cylinder piers on European work are often of very elaborate construction. The bridge on the Aa, at the crossing of the Russian Riga-Orel Railway, is supported on elegant cylinder piers, with molded caps, steel cutwaters, and are braced together with cylinders transversely. (Fig. 107.) This forms a very efficient construction, but so expensive to manufacture that it is usually replaced by bracing of struts and rods, as in Fig. 105, or by a metal diaphragm (Fig. 108). stiffened with angles.

Cylinders of water-tight construction and of large diameter may be used as coffer-dams where they are sunk into impervious strata,

or by sealing them with concrete around the bottom where they are placed upon smooth rock bottom. In the construction of lighthouses such cylinders have been placed upon clean rock bottom, through from 12 feet to 18 feet of water, and concrete deposited around the circumference of the base outside and inside to make them water-tight, after which they were pumped out and the foundation laid.

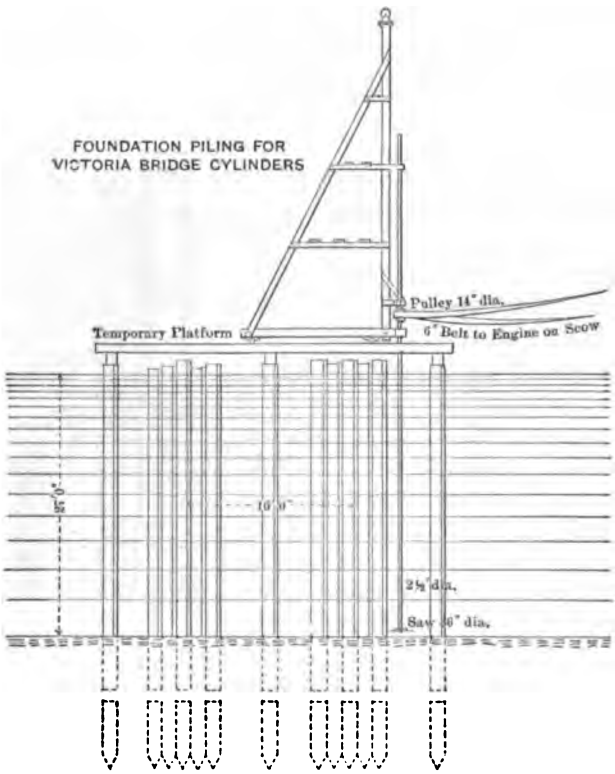


Fig. 106.—Circular Saw for Cutting off Piles under Water.

To withstand the pressure of any considerable depth of water the thickness and strength should be calculated and the construction carefully designed. Unless the depth of water exceeds 10 feet, or the diameter of tube exceeds 6 feet, the minimum thickness it is advisable to use will be sufficient for strength.

This refers only to quiescent pressure, and any shock must be carefully considered and taken account of, by interior bracing if necessary.

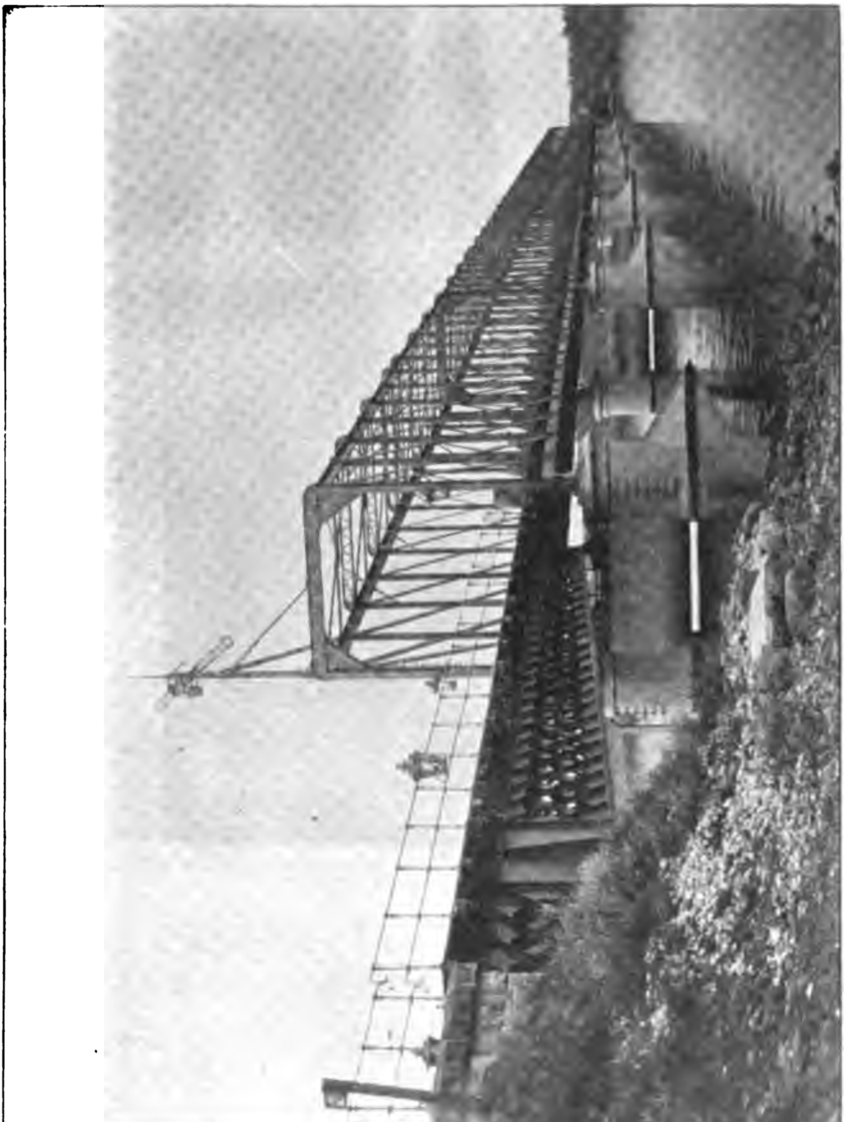


Fig. 107.—Cylinder-pier Bridge, Riga-Orel R. R., Russia.

The most thorough discussion of the strength of thin, hollow metal cylinders is given in "Elasticität and Festigkeit," by C. Bach. This considers the cylinder to have sides of a greater thickness than is true with pier shells, and having one radius given, the radius to the other side of the plate is found from the formula, the stress being variable from the inside to the outside of the plate.

For thin cylinders the stress may, without appreciable error, be assumed to be uniform over the cross-section of the plate, and the thickness t in inches be found from the formula, $t = .001rh$,

Fig. 108.—Cylinder Piers, with Diaphragm.

where r is the radius of the cylinder in feet, h is the depth of the water to the section in feet, and t in no case to be used less than $\frac{1}{4}$ inch in thickness.

This is on the assumption that the metal will stand 5000 pounds per square inch in compression with safety. For large cylinders, or for rectangular shells, girders and stiffeners or ties and struts must be added to prevent distortion.

The foundations for the great Forth Bridge, which were constructed under the direction of Sir John Fowler and Sir Benjamin Baker, required the use of various methods to reach solid bearing,

as the enormous weight to be carried required the most substantial piers obtainable.

The use of coffer-dams of metal for the Inch-Garvie piers is described by *Engineering:* The site of the two north or shallow piers being wholly submerged at high water, and about half in the case of the northeast and three-fourths in the case of the northwest pier, submerged also at low water, the preliminary work was tidal, and between spring tides no work could be carried on at all at this place. When it is considered how exposed the position was there—the work having to be carried on upon a narrow ledge of rock attacked by wind and waves from all sides—it will be understood that the progress could not be very rapid. The conditions of the

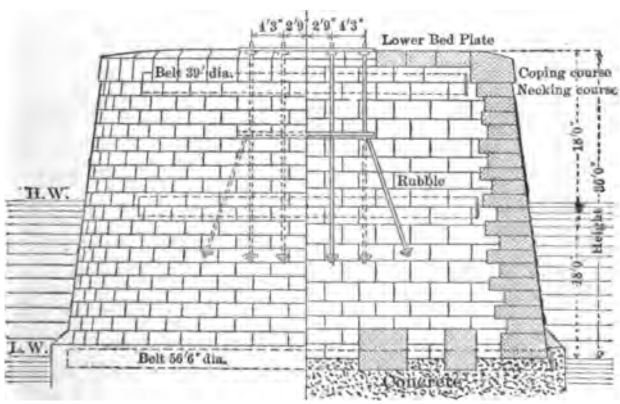


FIG. 109.—CIRCULAR GRANITE PIER AS FOUNDED BY COFFER-DAM, FORTH BRIDGE.

contract here required that the rock should be excavated in steps, and that the rubble masonry comprising the foundation of the circular granite piers (Fig. 109) should be bound by an iron belt 60 feet in diameter and 3 feet deep; the highest portion of the rock upon which this belt rested to be 2 feet below low water; the belt, or at any rate a part of it, to be brought down to form a protection for the foundation rubble masonry upon the lower steps.

It was therefore decided to cut a chase 8 feet wide (3 feet to the inside and 5 feet to the outside of the 60-feet circle) out of the rock where it was higher than 2 feet below low water, to make the 60-feet belt of three thicknesses of ½-inch plate and to carry the center plate downward, after it had been cut, in such a manner as to fit as nearly as possible the natural contour of the rock. (Fig. 110, *A*.) A light staging was, therefore, erected above high water, the correct

center of the pier placed upon it, and by means of a trammel-rod 30 feet in length, from the end of which a pointed sounding-rod was suspended, a correct reading was taken every 6 inches on the circumference of the 60-feet circle, after a diver had been around to clear out any loose stones lying in the line, or picking off any sharp points projecting. These readings were plotted and the center plates cut to it. In the meantime work had been done upon the chase, and,

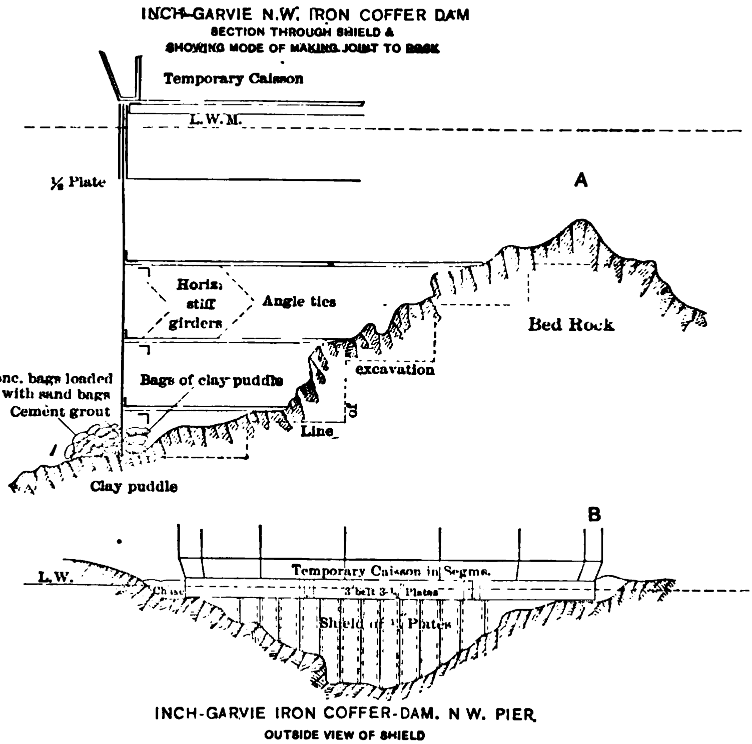


FIG. 110.—FORTH BRIDGE. METAL COFFER-DAM.

when nearly cut down to the right level, the belt was put together on the staging exactly above the site of the pier. The plates, projecting downward and forming the shield, were stiffened by I bars vertically over the butts, and where required to be carried down to a considerable depth, as in the case of the northwest pier, they were further stiffened by horizontal circular girders and stayed to the rock by bars of angle-iron. The whole belt was now riveted up, and when ready received two coats of red-lead paint, and was lowered down to position by means of hydraulic jacks. (Fig. 110, *B*.) The

top edge of the 3-feet belt was then leveled all round, and corrected where necessary. A heavy angle-iron $6'' \times 6'' \times \frac{7}{8}''$ ran round the inside of the 3-feet belt, and upon this was now set a single tier of temporary caisson, 10 feet in height, and consisting of fourteen segments of about 30 cwt. each in weight. This helped to keep the belt down to the rock, and a number of heavy blocks of stone were placed on the top of the caisson for the same purpose. A sluice door in the lower part was kept open to admit of the tide flowing in and out.

Steps were now taken to make good the joint between the 3-feet belt and the shield and the bed-rock. This was done in the following manner: A number of concrete bags, about $14'' \times 30''$, and 8″ to 9″ thick, were prepared and passed down to a diver, who laid them round the outside of the belt at a distance of about 4″. A second row was laid next round the outside of the first row, and tolerably close up, the space between the two being made up by clay puddle well stamped down. Any split or hole or crevice in the rock was also filled with clay. Upon these two lower rows other bags were now laid crosswise; upon these, two rows lengthwise, and a fourth row crosswise on the top, which was laid close up to the belt. This was done in sections of about 15 feet to 16 feet length all along the shield, but round the outside of the treble belt only two bags deep were laid. On the inside also a single row of clay bags, backed by a row of concrete bags, and loaded with stones, was laid round the complete circle. Cement grout, without intermixture of sand, was now prepared and passed down to the diver—but only at slack tide, high water, or low water—who lifted off one or more of the top bags and poured the grout into the narrow space left, until it overflowed. He then replaced the bag and proceeded to the next division, until all was done. Forty-eight hours were allowed to elapse for the setting of the cement; the sluice valve was then closed and the caisson pumped out gradually. When leaks were discovered the diver descended to examine the outside, and where necessary, he cut out some of the grouting and replaced it by new.

As it was not considered that this cement joint would be able to stand the full pressure of the tidal rise the coffer-dam was worked as a half-tide one, it having to be pumped out every tide as soon as the water had fallen below the top edge of the temporary caisson. In addition to the hydrostatic water pressure, the caisson had to stand the heavy seas thrown against it, whether coming from east or west. Under these circumstances it was often considered advisable not to pump out the coffer-dam, but leave the sluices open and allow the tidal flow free access. Under such conditions it will be easy to see

that during a season of bad weather, much delay could not be avoided, and though the work of excavation had been commenced in the summer of 1883 it was not till the middle of April of the following year that the first rubble masonry could be laid in this pier. In working the excavation no blasting was done within 1½ feet of the iron belt, but the rock was quarried up to within 6 inches and the rubble then built in at once. Any steps in the deeper cut portion were invariably at least twice as broad as they were deep. The deepest point to which the excavation had to be carried in this pier was 8 feet below low water.

The coffer-dam or caisson for the northwest pier, Inch-Garvie, was done in the same way precisely as described for the northeast, only that owing to the experience gained by the divers and other men engaged upon the work the progress was much more rapid.

In the northwest pier the depth of the shield was 15 feet below low water, and extended to nearly one-half of the circumference. There was, therefore, in addition to the vertical I bars which covered the butt joints of the shield-plates, three horizontal circular girders, carried at a distance of 4 feet 6 inches from each other; and from these a number of horizontal tie-bars with cross-bars at the ends were carried radially and level to the rock opposite and pinned to it, and afterward built into the solid rubble masonry. (Fig. 110, *B*.)

This mode of making the joint between the rock and the iron belt was simple and quite effective. Most of the leaks were due to natural crevices in the rock, running from the inside to the outside at a considerable depth. These were circumvented by building small clay dams round, and leading the water by a chute to the pump. Leaks were also caused by the action of heavy waves running up to the temporary caisson at low water with great violence, and shaking the whole fabric.

The whole of the northeast pier was built in a half-tide caisson, as the work was not pressing; but in the case of the northwest pier, as soon as the rubble masonry inside had been brought up to a low-water level a second tier of temporary caisson was added, and the work could then be carried on at all states of the tide. While tidal work was carried on in these two coffer-dams the amount of water which had to be pumped out every tide was 250,000 gallons in the one case and 340,000 in the other. The time occupied was 50 to 55 minutes, but work was, of course, commenced as soon as the higher parts were laid dry. For pumping out smaller quantities of water collected through leaks, pulsometers or small centrifugal pumps were used.

An exterior view of the work is shown in Fig. 111, and while the

method was successful and worthy of much study, the expense would only be justifiable where the metal would be retained as part of the permanent foundation, which was the case on this work.

In many cases such a shell could be designed of the proper size for the footing course, and after use as a coffer-dam in obtaining the foundation it could be filled with concrete and serve as a base for the pier. Being made in sections vertically, portions projecting above low water could be removed and used on still other piers.

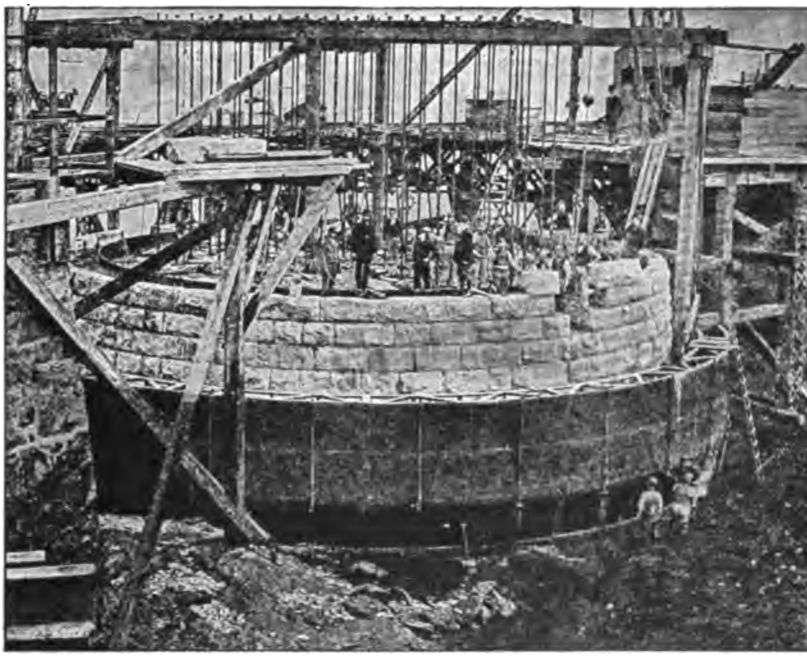


FIG. 111.—FORTH BRIDGE. CIRCULAR GRANITE PIER AND METAL COFFER-DAM.

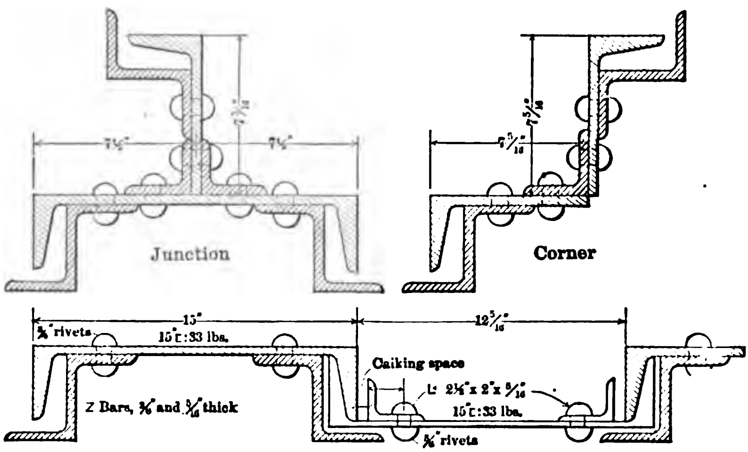
Metal sheet-piles were used on some harbor work at Cuxhaven Harbor, Germany. These were hollow metal sheet-piles of elongated, elliptical sections; and, after being driven, were filled with concrete.

Metal sheet-piling is being largely used in the United States, frequently of Friestedt patent type. This is described in the *Engineering News* as follows:

" In many foundation works, particularly in quicksand and wet ground, the ordinary timber sheet-piling cannot be used to good advantage, and on works of this kind the use of steel sheeting is now being introduced. Fig. 112 represents a style of sheet steel-piling

which has been used for the sheeting of foundations, mine-shafts, and also for coffer-dams, locks, etc.

"The cut clearly shows the construction of the sheeting and also two arrangements for corner construction. The piles consist of ordinary 15-inch, 33-pound rolled channels (with metal $\frac{3}{8}$ inch thick), each alternate channel having riveted to it two steel Z bars, forming grooves to receive the flanges of the adjacent channels. These bars are $4'' \times 3'' \times 3''$, $\frac{3}{8}$ inch thick. In this way the piles are so firmly interlocked that a line of sheeting will resist heavy pressure without the aid of shoring or bracing. In water the joints are soon calked by the acccumulation of mud, sand, or dirt, but if the water should be very clear, a little sawdust, paper, pulp, manure, etc., may be thrown



STRAIGHT SHEETING.

Fig. 112.—Friestedt Sheet-piling.

in near a leak, which will soon be sealed. In the Mississippi River coffer-dam at St. Louis, noted below, the alternate piles had two angle-irons $\frac{5}{16}'' \times 2\frac{1}{2}'' \times 2''$ riveted on (as shown in the cut) to form calking grooves in which strips of wood might be fitted. These, however, have been found unnecessary and have been omitted on part of the work.

"This steel sheeting has been used for sinking mine-shafts through quicksand, and for the foundations of the Union Traction Building at Cincinnati and the new Railway Exchange Building at Chicago. The latter will be a seventeen-story 'sky-scraper' office building, at Jackson Boulevard and Michigan Avenue, and the site will be excavated to a depth of nearly 30 feet. As there is a bed of quick-

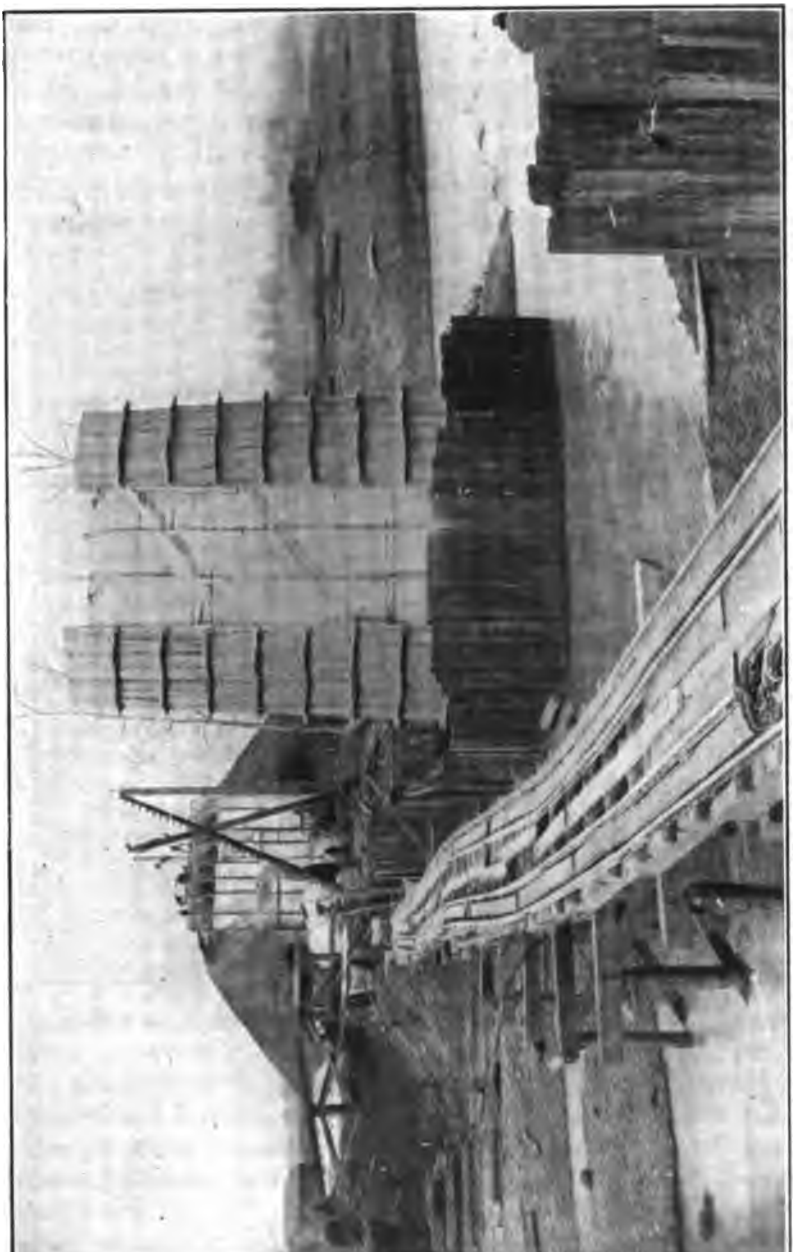


FIG. 113.—COFFER-DAM, C. B. & Q. RY., WITH FRIESTEDT PILING

sand underlying the site, it was decided to completely surround it with a steel sheeting, driven to a depth of 30 feet, which will resist the pressure from the outside and so prevent flow or caving which might injuriously affect adjacent buildings. This work is now in progress. An ordinary pile-driver with a 2000-pound hammer is used, the head of the pile being fitted with an iron cap having a wooden cushion and a wooden striking-block. The sheeting has been used also

FIG. 114.—LACKAWANNA STEEL SHEET-PILING. M. D. & S. R. R. BRIDGE.

to form the coffer-dam for the new power-house of the Union Electric Light & Power Co., at St. Louis (*Eng. News*, Nov. 6 and Dec. 18, 1902). The location is on the bank of the Mississippi River, and a coffer-dam, 40′×360′, was built of the steel piles, 50 feet long, braced only by cross-walls dividing it into panels or sections, 40′×60′.

"The piles are rolled of any length up to 60 feet, and can be spliced for longer lengths. In many cases it can be pulled up and used over again, and discarded material will have a high scrap value."

The cross-section of piling is shown in Fig. 112, while Fig. 113 shows some work on the C., B. & Q. Ry., on which it has been used. Whatever the class and form of material it may be decided to use in securing a foundation by the coffer-dam method, the temporary construction should be so related to the permanent foundation that as much as possible of the material used and labor employed will be of service in the finished structure.

The Lackawanna steel sheet-piling is another type that is very extensively used in constructing coffer-dams, and its use is exactly similar to that already described. The construction of the founda-

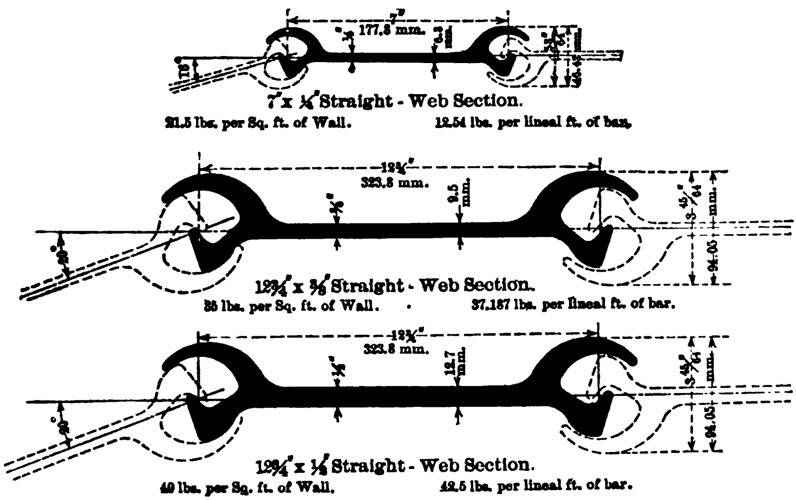


FIG. 115.—LACKAWANNA STEEL SHEET PILING.

tions for a bridge over the Ocmulgee River for the Macon, Dublin & Savannah Railroad is shown in Fig. 114, where it was employed by D. B. Dunn, Chief Engineer of the road. The river bed was underlaid by stiff clay 20 feet underneath the bed of sand. The sheet-piling was used in 45-foot lengths spliced from one 30-foot piece, and one 15-foot piece, the driving being done without any trouble in the regular shape shown in the photograph. The piling inside the coffer-dams was cut off 8 feet below the bed of the river, and concrete filled in about to the water level, or about to the top of the 30-foot piece of the sheet-piling, from which point the pier was stepped in and carried up in ordinary forms. This sheet-piling is of three forms: the straight-web type, Fig. 115, made in sizes shown in Table XIV; the arched-web type, Fig. 116, and is made of the sizes shown in

Table XIV. For heavier work the center-flange type shown in Fig. 117 may be used, and the sizes of this are given in Table XIV. This piling may be pulled out and used over again by means of a hole in the

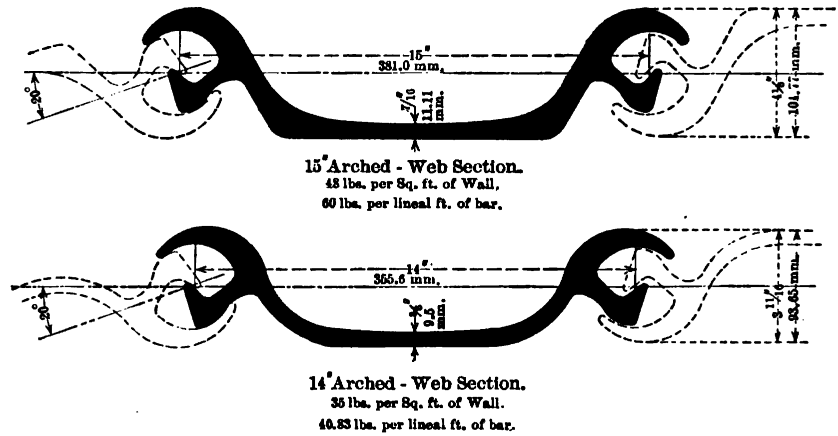


FIG. 116.—LACKAWANNA STEEL SHEET PILING.

top end, and into which the shackle from a set of falls may be attached. The straight-web type can be used satisfactorily for shallow work, but where there is considerable pressure, either the arched web or the

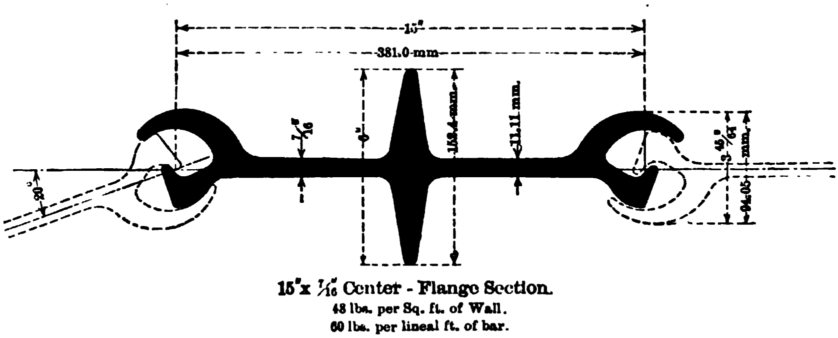


FIG. 117.—LACKAWANNA STEEL SHEET PILING.

center-flange type must be used. Should it be desired to leave the piling in where it is not protected with concrete, the material must be of sufficient thickness to stand rusting or the corrosive action of sea water. The efficiency of this piling is shown in Table XV.

TABLE XIV.—WEIGHT OF LACKAWANNA SHEET PILING.

Sections.	Weight per Lineal Unit of Bar.		Weight per Unit Area of Wall.	
	Per Lineal Foot in Lbs.	Per Lineal Meter in Kilogs.	Per Square Foot in Lbs.	Per Sq. Meter in Kilogs.
15" Center-flange.....	60	89.29	48	234.34
15" Arched-web......	60	89.29	48	234.34
14" Arched-web......	40.83	60.76	35	170.90
½" Straight-web......	42.5	63.25	40	195.32
⅜" Straight-web......	37.187	55.36	35	170.90
¼" Straight-web......	12.54	18.60	21.5	104.97

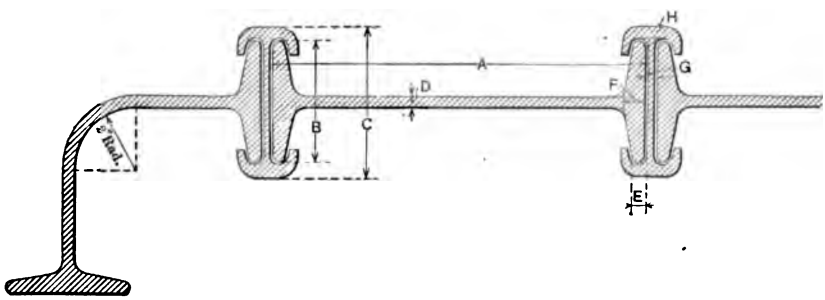


FIG. 118.—JONES AND LAUGHLIN STEEL SHEET PILING.

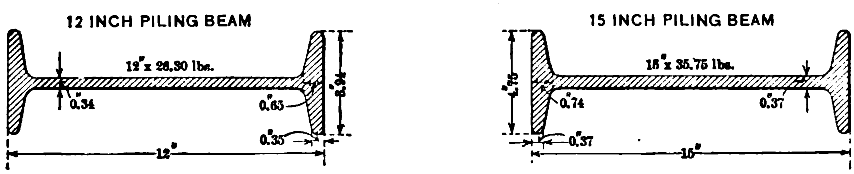


FIG. 119.—J. & L. SHEET PILING BEAMS.

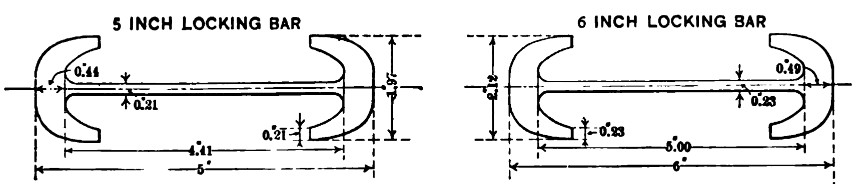


FIG. 120.—J. & L. LOCKING BARS

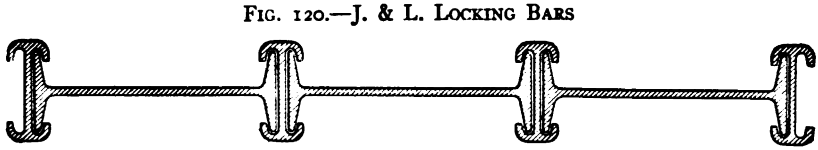


FIG. 121.—J. & L. SHEET PILING BEAMS.

Another very satisfactory type of metal sheet-piling is that manufactured by Jones & Laughlin. This is shown in Figs. 118, 119, 120, and 121. The dimensions and properties of this piling are shown in Tables XVI, XVII, XVIII, and XIX. This piling is shown in use

FIG. 122.—BRIDGE FOUNDATION WITH J. & L. SHEET PILING.

in constructing a bridge pier foundation in Fig. 122. On account of it being made out of standard I beams, the piling can be pulled easily by holes in the top, as shown in the cut, and if there is no further use for it on any other work of this kind, the beams may be used in structural work, or any construction where they will fit.

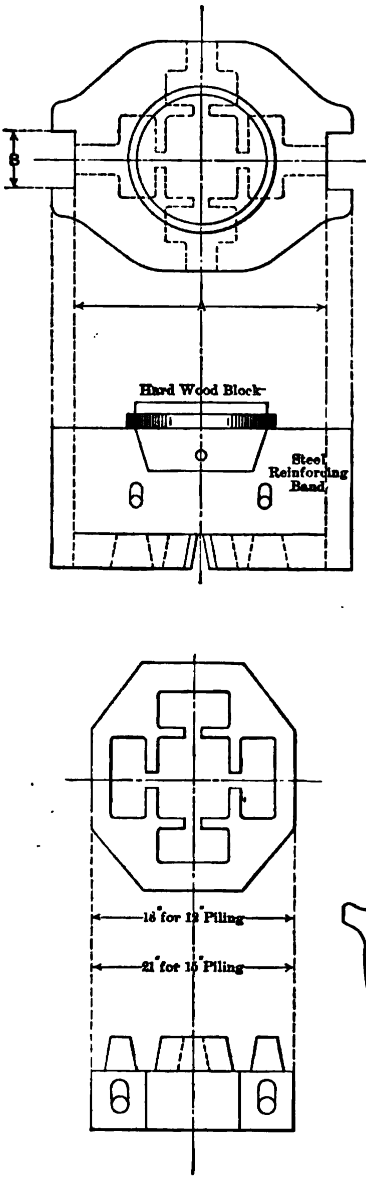


FIG. 123.—FOLLOWER CAP FOR J. & L. PILING.

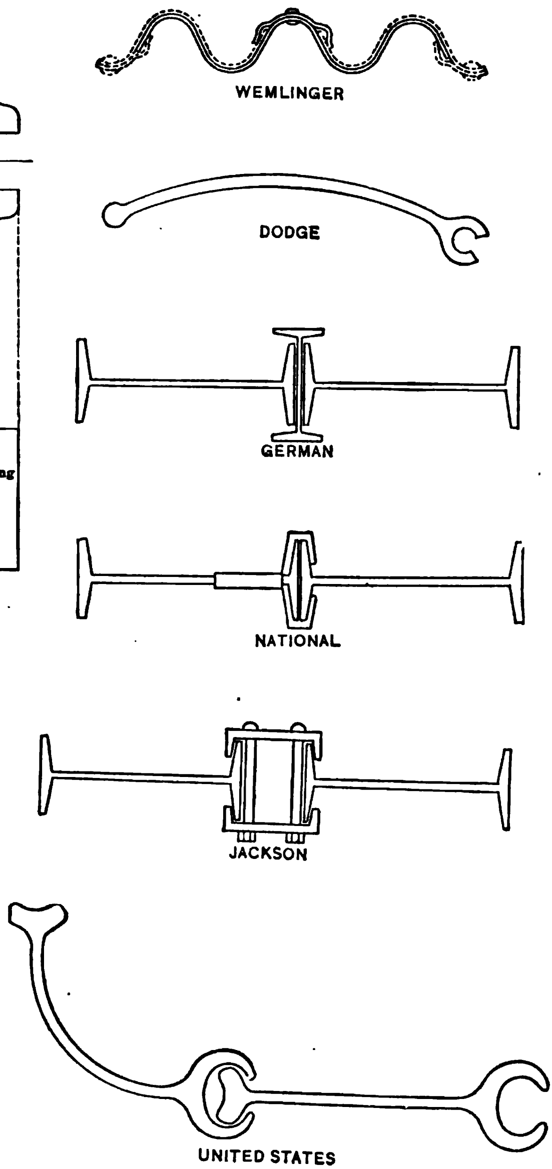


FIG. 124.—OTHER FORMS OF STEEL SHEET-PILING.

The lock bars would make very satisfactory re-enforcing for re-enforced concrete work. In driving these piles, a follower cap similar to that shown in Fig. 55 may be used, the details of which are given in Fig. 123.

Various otner types of steel sheet-piling are manufactured, some of which are shown in Fig. 124, but, as the principle is the same in all of them, they will not be further discussed.

TABLE XV.—EFFICIENCY OF LACKAWANNA STEEL SHEET PILES IN TRANSVERSE STRENGTH.

As represented by Coefficient of Strength per pound of metal in a pile beam 12 inches long.

Type.	Straight-web.			Center-Flange.	Arched-web.	
Size.	7" × ½"	12¾" × ⅜"	12¾" × ½"	15" × 7/16"	14" × ⅜"	15" × 7/16"
Least radius of gyration of single section	0.373	.760	0.729	1.01	1.150	1.290
* Modulus of interlocked sections, c. to c. of web.	0.575	4.040	4.210	6.25	8.890	14.110
Modulus of single section.	0.567	4.000	4.120	5.73	7.610	11.800
Modulus per inch of width of single section	0.081	0.313	0.323	0.382	0.543	0.786
Efficiency in transverse strength of single section, weight not considered	†1.00	3.86	3.98	4.72	6.70	9.70
Coefficient of strength, factor of satfety 3, fiber stress taken at 20,000 lbs. per sq.in. $\frac{2}{3}\left[20{,}000 \times \text{modulus of single section}\right]$	7560.00	53333.33	54933.33	76400.00	101466.66	157333.32
Wt. in lbs., lin. foot of pile	12.54	37.187	42.50	60.00	40.83	60.00
sq. foot of wall	21.50	35.00	40.00	48.00	35.00	48.00
Coefficient of strength of single section per pound of metal = $\frac{\text{Coeff. of strength}}{\text{wt. of lin. ft. of pile}}$	602.87	1434.19	1292.54	1273.33	2485.10	2622.22
Efficiency considering both transverse strength and weight, or the coefficient of strength of single section per lb. of metal.	†1.00	2.37	2.14	2.11	4.12	4.34

* It is safer not to use the interlocked modulus in designing steel sheet-pile structures, unless, as in permanent constructions, the open spaces remaining in the interlocked joints of the piling, after driving, are filled with cement grout before pressure is brought upon the wall; thus preventing movement of the interlocking members of the joint and also preventing a change in position of the neutral axis of the wall.

The modulus of the single-pile section, while somewhat lower, is more conservative, as the neutral axis of such single section is not so subject to change of position after the pile is installed in the wall.

† Taken as the basis for comparison.

TABLE XVI.—JONES AND LAUGHLIN STEEL SHEET PILING

No.	Size, Inches.	Weight per Sq. Ft. Lbs.	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*
1	12×5	35.00	12	3.94	5	0.344	0.35	0.65	0.21	0.443
2	12×5	36.25	12	3.97	5	0.375	0.35	0.65	0.21	0.443
3	15×6	37.20	15	4.75	6	0.375	0.375	0.74	0.23	0.489
4	15×6	39.75	15	4.81	6	0.4375	0.375	0.74	0.23	0.489
5	15×6	42.25	15	4.87	6	0.500	0.375	0.74	0.23	0.489

TABLE XVII.—PROPERTIES OF J. & L. STEEL SHEET PILING BEAMS

Section No.	Depth of Beam, Inches.	Weight per Linear Foot-pounds.	Area of Section Square Inches.	Thickness of Web Inches.	Width of Flange Inches.	Moment of Inertia Neutral Axis Perpendicular to Web at Center.	Moment of Inertia Neutral Axis Coincident with Center Line of Web.	Radius of Gyration Neutral Axis Perpendicular to Web at Center.	Radius of Gyration Neutral Axis Coincident with Center Line of Web.	Section Factor Neutral Axis Perpendicular to Web at Center.	Section Factor Neutral Axis Coincident with Center Line of Web.
1	12	26.30	7.72	0.34	3.94	167.76	4.43	4.67	0.76	27.96	2.25
2	12	27.60	8.10	0.38	3.97	172.10	4.56	4.61	0.75	28.68	2.30
3	15	35.75	10.50	0.38	4.75	358.16	8.52	5.84	0.90	47.75	3.59
4	15	39.00	11.44	0.44	4.81	375.03	8.91	5.71	0.88	50.00	3.70
5	15	42.25	12.37	0.50	4.87	391.92	9.31	5.62	0.87	52.25	3.82

TABLE XVIII.—PROPERTIES OF J. & L. LOCKING BARS

Section No.	Depth of Locking Bar Inches.	Weight per Linear Foot. Pounds.	Area of Sections Square Inches.	Thickness of Web Inches.	Moment of Inertia Neutral Axis Perpendicular to Web at Center.	Moment of Inertia Neutral Axis Coincident with Center Line of Web.	Radius of Gyration Neutral Axis Perpendicular to Web at Center.	Radius of Gyration Neutral Axis Coincident with Center Line of Web.	Section Factor Neutral Axis Perpendicular to Web at Center.	Section Factor Neutral Axis Coincident With Center Line of Web.
1 & 2	5	9.75	2.87	0.21	10.50	0.64	1.91	0.47	4.20	0.64
3, 4 & 5	6	12.25	3.61	0.23	18.42	1.11	2.26	0.55	6.14	1.03

TABLE XIX.—PROPERTIES OF COMBINED SECTIONS FOR PURPOSE OF COMPARISON

Section Number.	Size, Inches.	Weight per Square Foot of Assembled Area. Pounds.	Total Sectional Area Assembled Section. Square Inches.	Width of Joint Over All. Inches.	Moment of Inertia Neutral Axis Coincident with Center Line of Web.	Radius of Gyration Neutral Axis Coincident with Center Line of Web.	Section Factor Neutral Axis Coincident with Center Line of Web.
1	12×5	35.0	10.59	5	6.07	1.07	2.77
2	12×5	36.25	10.97	5	6.11	1.05	2.80
3	15×6	37.20	14.11	6	12.85	1.24	4.24
4	15×6	39.75	15.05	6	13.45	1.21	4.28
5	15×6	42.25	15.98	6	14.14	1.18	4.35

CHAPTER IX

CYLINDERS AND CAISSONS

THE preceding chapter discussed to some extent the use of tubes filled with concrete for cylinder piers, and the details of their manufacture, and placing will now be more fully treated.

It is customary in many bridge shops to make the vertical distance between rivet lines of tubes exactly 5 feet for all diameters, thus making it possible to carry in stock plates for the manufacture of highway piers, these plates being 62½ inches in width and of varying lengths equal to the circumference of various-sized tubes, plus the 2½ inches for lap, as is also used for vertical lap.

These plates are then laid out; that is, the location of the rivet holes marked by standard wooden templets; after the plates are punched they are rolled in boiler bending rolls, and assembled in as great lengths of tubes as can be handled on the cars for shipment, or hauled from the railroad to the bridge site, or, in the case of large tubes, in as long pieces as can be erected. This is very often only one section in length with the vertical seam riveted.

The sections of tubes with lap-joints are alternately large and small, the diameter differing by a little over twice the thickness of the plates, so as to telescope (Fig. 125); the small section in every case having the outside diameter equal to that specified for the cylinders of the piers.

The tubes also have holes to which the bracing between them is attached, which usually consists of struts and sway-rods.

Railroad piers of this type are generally manufactured from material which is rolled to conform to a particular specification, and the sheets can be ordered of such widths as will best suit each particular case. The tubes are made with butt joints and with much closer rivet spacing than is used for highway piers; that is, in place of 4- to 5-inch spacing, they are spaced usually from 3 to 4 inches centers.

Frequently angle-rings are riveted around the tops of the tubes, and sometimes around the bottom to stiffen them, and in some cases

TABLE XX.—TUBE WEIGHTS AND QUANTITIES.

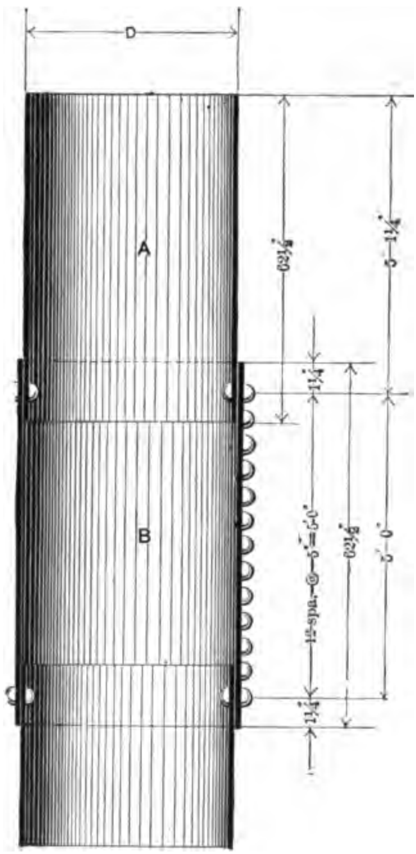
Diameter of Tube.		Thickness and Order Length. ¼		5/16		⅜		½		All Quantities Below for Two Tubes or One Pair. Weight of one 5-ft. 0-in. Section. ¼	5/16	⅜	½	Cu. Yds. 1 Foot, Vertical.	Weight ¼-in. Cap.	Weight ⅜-in. Cap.
Ins.		Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.							
15	A	4	1	4	0½	.....		.....		448	560	...	...	.090	30	45
	B	4	2½	4	2½											
18	A	4	10½	4	10	.....		.....		532	664	...	...	.130	40	60
	B	5	0	5	0											
21	A	5	8	5	7½	.....		.....		616	770	...	...	.178	55	80
	B	5	9½	5	9½											
24	A	6	5	6	5	.....		.....		700	874	...	...	.232	70	105
	B	6	7	6	7											
27	A	7	2½	7	2½	.....		.....		780	980	...	...	.296	85	130
	B	7	4	7	4½											
30	A	8	0	8	0	7	11½	.....		864	1084	1302	...	.364	115	165
	B	8	1½	8	2	8	2									
33	A	8	9½	8	9½	8	9	.....		948	1190	1428	...	.440	135	195
	B	8	11	8	11½	8	11½									
36	A	9	7	9	6½	9	6½	9	6	1032	1290	1554	2080	.524	155	220
	B	9	8½	9	8½	9	9	9	9½							
39	A	10	4½	10	4	10	4	10	3½	1116	1394	1680	2242	.614	180	265
	B	10	6	10	6	10	6½	10	6½							
42	A	11	2	11	1½	11	1½	11	1	1200	1500	1806	2408	.712	210	310
	B	11	3½	11	3½	11	4	11	4							
45	A	11	11	11	11	11	11	11	10½	1284	1604	1926	2576	.820	240	355
	B	12	1	12	1	12	1	12	1½							
48	A	12	8½	12	8½	12	8	12	8	1364	1710	2052	2744	.930	270	400
	B	12	10	12	10½	12	10½	12	11							
54	A	14	3½	14	3½	14	3	14	2½	1532	1920	2304	3080	1.178	345	510
	B	14	5	14	5½	14	5½	14	6							
60	A	15	10½	15	10	15	10	15	9½	1700	2124	2556	3408	1.454	425	630
	B	16	0	16	0	16	0½	16	0½							
66	A	17	5	17	5	17	5	17	4½	1864	2336	2802	3746	1.758	510	760
	B	17	7	17	7	17	7	17	7½							
72	A	19	0	19	0	18	11½	18	11½	2032	2546	3056	4080	2.094	605	900
	B	19	1½	19	2	19	2	19	2½							
78	A	20	7	20	6½	20	6½	20	6	2200	2750	3306	4416	2.458	710	1055
	B	20	8	20	8½	20	9	20	9½							
84	A	22	1½	22	1½	22	1½	22	2	2364	2960	3554	4746	2.850	830	1235
	B	22	3	22	3½	22	3½	22	2							

NOTE.—To get weight of *shells* for one pier divide height of pier by 5, multiply by weight of section, add in caps (which include cap-lugs).

Per cent of rivets: For 15″×¼″, 3%; 48″×5/16″, 2½%; 84″×½″, 2%. For other sizes use value for smaller size.

are used to attach cap-plates to cover the tops, but it is better practice to carry a richer concrete above the top several inches and round it off around the edge to shed water.

The erection of the tubes to form piers is very often left to careless or incompetent foremen; but while the design of the metal work may be easily carried out to conform to some standard, each case of erection requires special treatment, and should be placed in careful and experienced hands.



NOTE:- Make tubes of 62½" sheets, plus one narrower ring.
Horizontal spacing about 5 In.
Locate connections from horizontal gauge lines.

FIG. 125.—ORDER DIAGRAM CYLINDER PIERS.

Where the bottom is soft, the material should be removed to a harder stratum or to a considerable depth before the tubes are set in position, the placing of them being carried out by means of a gin-pole, a derrick, or a gallows-frame, as may be found necessary.

Should the hard stratum not be reached before the tubes are set, the excavation must be proceeded with by excavating inside by hand; or if this becomes impossible, by using machinery, such as a small orange-peel dredging-bucket which is small enough to go inside the tube. Very soft material may be removed by pumping it out with a centrifugal pump. Where the material is too hard for the tube to sink of its own weight as the excavation proceeds, then it must be weighted in some way. This has been done by add-

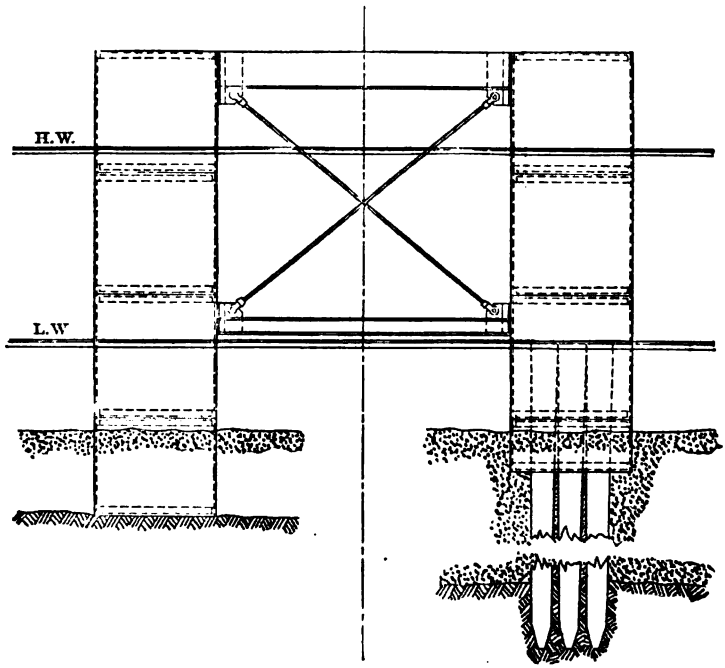


FIG. 126.—DESIGN FOR CYLINDER PIERS.

ing a section or so to the tube, or by building a platform around the top and loading it with the excavated material. Where even then the cylinder sticks, a water-jet used around the outside may overcome the skin friction and assist the sinking.

The tubes need only be sunk to such a depth as will insure against scour, as shown in Fig. 126, unless by going a reasonable additional distance hard enough material may be reached, so that within the area of the cylinders of one pier sufficient bearing capacity will be had.

Otherwise piling in sufficient number must be driven inside each tube to carry the load, and these must be cut off near enough to low-

water line to insure their keeping constantly wet to prevent rot. Enough space must be left between the piles, and between the piles and the metal shell, in which to tamp concrete.

Another plan very often used for supporting cylinder piers on soft ground is to drive piles to carry a grillage on which to set and fasten the tubes. The grillage may be drift-bolted to the piles, which have been cut off to a level, or the grillage may be fastened to the bottom of the tube and the whole then lowered onto the piles, guide-piles around the sides being used to insure the proper placing and to anchor the tubes in position. With such a foundation, riprap stone should always be used for protection from the current of the stream, and riprap should always be used wherever there is any possibility of scour.

Where the pier is placed in the current a crib should be placed around it, if protection is necessary, and the space between the crib and the cylinders filled with riprap. (Fig. 127.) Enough riprap should also be placed outside of the crib to protect that from scour.

Where there is solid rock on which to place the cylinders, they should be anchored to it some way. In shallow water this is usually done by placing a coffer-dam around the pier so that the rock can be leveled off whatever is necessary, and anchor-bolts used to anchor the cylinders in place, the anchor-bolts passing through lugs on the bottom, or through the legs of an angle-rim. If the rock is at all uneven and it is impossible to lay it bare to do the leveling off, then the bottom of the tube should be cut to fit the irregularities as they are learned from soundings. Anchors may be placed by drilling into the rock and setting old railroad rails into the drill holes, allowing the rails to project up several feet into the cylinders. Then when the cylinders are placed, concrete may be deposited in them under water by some of the approved methods.

It is very often advisable to put a crib around a pier on solid rock in order to protect it against the force of the current, driftwood, and floating ice.

Considerable mention has already been made of piers being sunk by dredging out inside either tubes, such as have just been treated of, or larger metal piers, such as were used for the Hawkesbury Bridge. In the United States, owing to the fact that timber is so plenty and cheap, it has been the practice to use piers similar to those of the Hawkesbury Bridge with the caisson and crib of timber instead of metal. An example of this type of pier is shown in Fig. 128, which was designed by the author for a draw pier for the Northern Pacific Railway Company, the outside diameter of the octagonal crib being

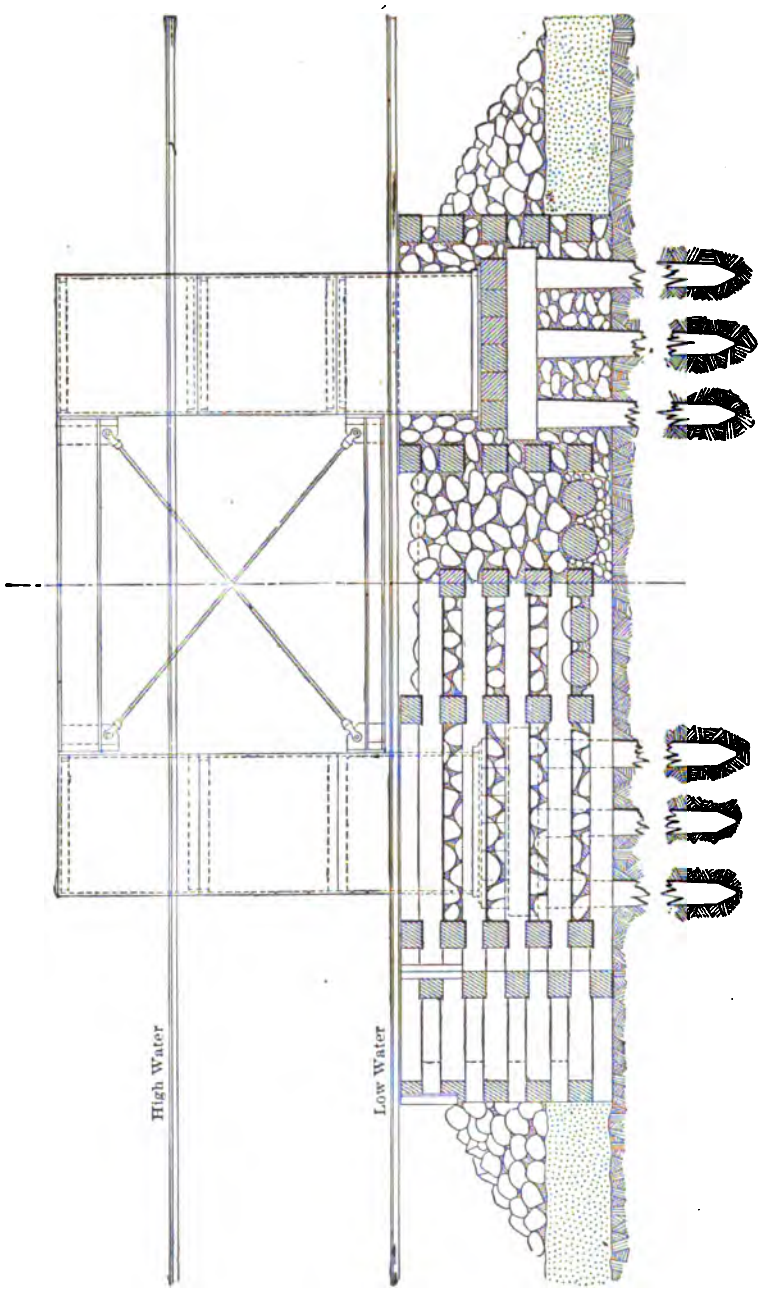


FIG. 127.—CYLINDER PIER WITH CRIB.

38 feet 4 inches, and the total height of the pier being 84 feet above the cutting edge. The caisson proper is built up solid of timber with one cross-cutting edge, which was deemed advisable on account of

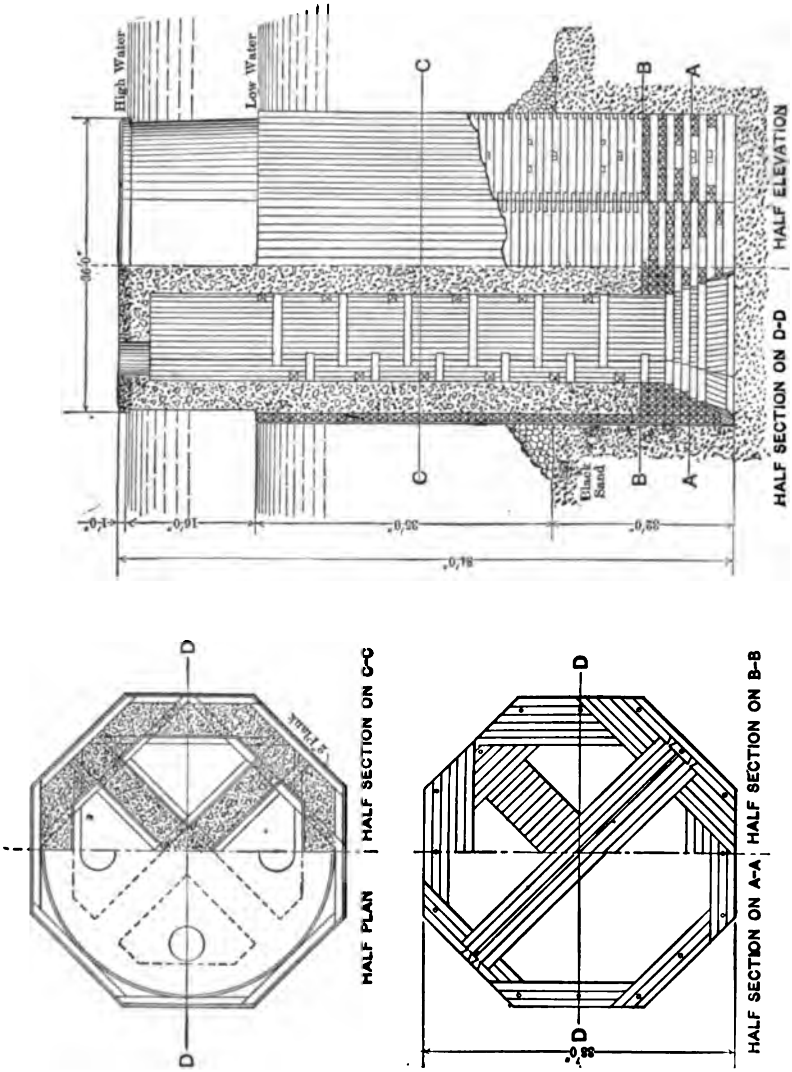


FIG. 128.—PIER FOR NORTHERN PACIFIC RY.

the large diameter of the pier. The height of the caisson to the roof is 8 feet 6 inches, while the roof consists of three layers of 12×12 timbers, each of which is thoroughly calked and pitched. The sloping sides of the caisson walls are stepped to avoid the bulging pressure

during sinking, instead of straight as has been customary on work of this character. This is a detail of construction which has been developed by J. A. L. Waddell, M. Am. Soc. C.E., on large work of this character which he has completed and which will be mentioned more fully in this chapter.

Above the caisson a timber crib is carried up to low water, and the outside of the caisson and of this crib are thoroughly calked, covered with tarred ship-felt and planked, the ship-felt and plank serving as additional protection for water-tightness and as protection against the teredo, while the planking also very materially strengthens the crib. Above the top of the crib a temporary crib coffer-dam is used to exclude the water while laying the concrete, although it is possible, on account of the 16 feet range of tide, to lay most of it in the dry with ordinary concrete forms.

Instead of dredging out the material with sand-pumps of the ordinary type or with dredging-buckets, the Hendy hydraulic elevators No. 5 are used, Fig. 129, with a water pressure of 125 pounds. An elevator of this sort can be shifted around and the material dredged out of different parts of the caisson, so as to cause the pier to sink evenly. But, should it be impossible, for any reason, to keep the pier plumb, this can be corrected by the permanent water-jets, as indicated in the sections of the pier. Some of these jets come out at the cutting edge and some of them along the sides of the pier. It has been found possible, however, to accomplish better results by using separate jets at such points on the outside as may be necessary. Should the jets fail to correct the trouble, then it is necessary to resort to the use of dredging on the outside, although this is not very often resorted to. With a pier of this sort the caisson is constructed on shore up to such a height as will bring the top of the timber above the water after it is launched and floated. (Fig. 130.) It should then be carried up rapidly enough, so that as the concrete is filled in it will be kept above the water until the pier is landed on the bottom as it is sunk. Only sufficient concrete of course can be put into the pier to keep it sinking properly, and with the crib calked and water-tight, it can be pumped out and the concrete laid in the dry. The concrete specified for this work was one part of Portland cement, three parts of sand, and five parts of screened and washed gravel. A facing of mortar 1 to 3, with a thickness of 1½ inches, is specified for the facing, and the entire coping of 1 to 2 mortar reinforced with wire-netting and railroad rails, so as to carry machinery at any point.

Upon the pier reaching the proper depth and the wells being

FIG. 129.—HENDY HYDRAULIC ELEVATOR.

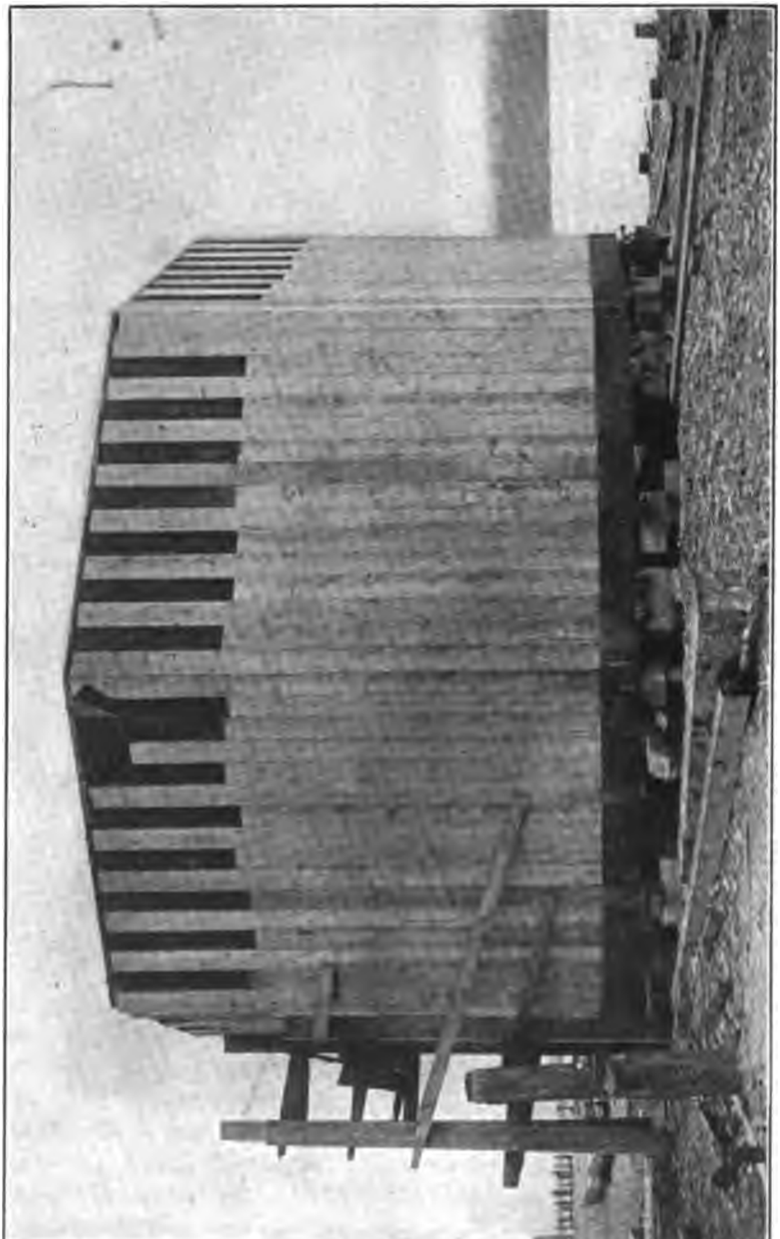


FIG. 130.—CAISSON NORTHERN PACIFIC PIER.

cleared of all material, they are filled to about 40 feet above the base with concrete, which is first put into the caisson through the water until the caisson is sealed. Then the wells are pumped out and the balance of the filling placed in the dry.

Piers of very similar design were used for the bridge over the Fraser River, at New Westminster, B. C., under J. A. L. Waddell, consulting engineer. There were five piers of this type, some of them having a total height exceeding 125 feet. The endeavor was made in the design of the caissons and cribs to deposit as nearly all of the concrete as possible in the dry. All the timber work was thoroughly calked and, on account of the calking on the roof timbers being on the opposite side from what was necessary for resisting the pressure, pitch was used on all the seams, consisting of crude resin mixed with a sufficient quantity of tallow so that it would be stiff and yet not too brittle. This was mixed in a kettle and the ordinary pitch vessel used for pouring it into the seams. In one of the first piers constructed only the seams of the 2-inch sheathing were calked, and there was considerable leakage; so that all of the seams in the other cribs were calked, including the 12×12 timbers. The experience had on this work points to the advisability of using two threads of oakum in all this calking. Where the jet-pipes pass through the roof of the caisson it was found necessary to seal the openings with pitch or a rich grout, and the entire surface of the rock was covered with an 8-inch layer of rich concrete, it being placed while the deck was above water-level. On account of the trouble had from leakage around the pipes, the later piers had these carried down through the well holes. In building up the cribs the timbers were lapped at the corners and well drifted, and were found to be stronger than the ordinary method of dapping. The solid timber portion of the work was drift-bolted at every crossing, thus tying the corners permanently together. When a small amount of penetration had been gotten on piers Nos. 4 and 5, the concrete work about the wells was carried above the surface of the water, and all timber work about the wells except the sheathing omitted. With pier No. 3, however, a great deal of trouble was found in holding the crib plumb, and it was never possible to carry the concrete to the level of the water-surface. Consequently solid well timbers were carried up to the very top of the crib. In sealing the wells the amount was regulated by the height of the crib. This was decided upon at 75 feet for pier No. 5, and 50 feet of sealing was used; while for piers Nos. 3 and 4, lifts of 55 and 65 feet were used respectively, this sealing being allowed to set a week in each case. For any work of this character the caisson timbers should be very thoroughly

fastened together and ordinary ship-building methods employed to the extent of using a great many through-bolts riveted down on clinch-rings. With a circular or octagonal crib the cutting edge should be spliced together at all points sufficiently strong to make a tension-ring about the bottom to take up the bulging pressure.

The 232-foot plate-girder drawbridge for the city of New Westminster, B. C., over the north arm of Frazer River, to Lulu Island, is supported on a center pier constructed by the author by sinking a crib made of 12×12 timbers to a depth of 33 feet below low water.

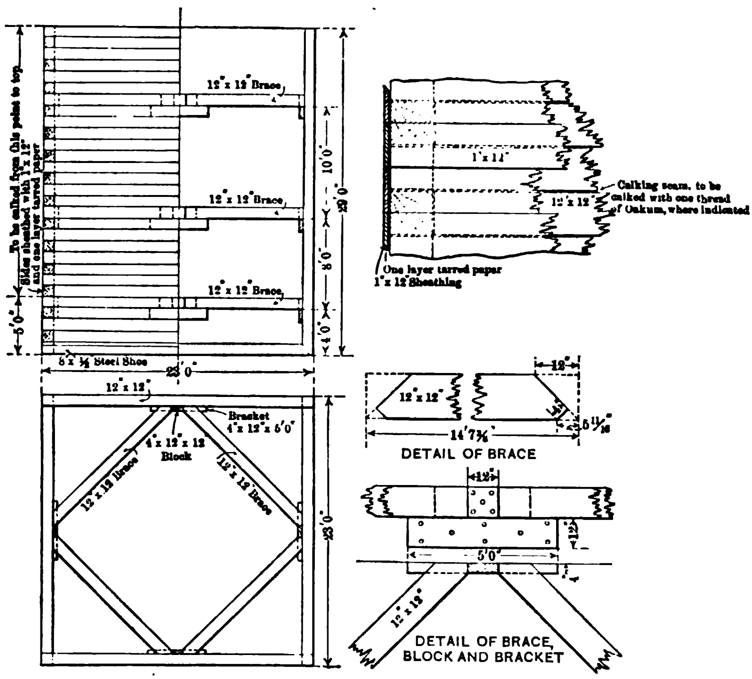


FIG. 131.—CRIB FOR LULU ISLAND BRIDGE.

This crib, shown in Fig. 131, was 23 feet square, and braced with 12×12 timbers as shown. The borings showed that the crib to the full depth was to be sunk through black sand, and as the Consulting Engineers would not approve the use of sheathing on the outside of the crib, or the use of a steel cutting edge, these were omitted. On account of the small size of the crib, the sand-boxes were placed on the outside to give the required weight for sinking. The plant available for sinking this crib comprised, among other necessary equipment, a 6-inch sand-pump, and as the material was believed to

be, according to the borings, entirely of sand, this was used for pumping out the material to allow the crib to sink. Owing to the omission of the outside sheeting, the exterior of the crib was very rough, and the coefficient of friction very high. The lack of a metal cutting edge caused the material to come in under the bottom of the crib in much greater quantities than ordinary, and the sinking proceeded very slowly. Gravel was encountered in considerable quantity and added very much to the difficulty of putting down the crib; and considerable trouble was experienced in handling the long suction of the pump when it was necessary to clean it out. The suction pipe was a rubber suction hose with smooth interior, but this quickly cut out with the sharp sand and gravel and had to be frequently replaced. No foot-valve was used, and when the suction was pulled up by the derrick for cleaning, the pump was reprimed by driving a wood plug into the end of the pipe; this plug being fastened up with a short line so that when the pump and pipe were filled with water, and the suction dropped into the crib, the plug would jerk out soon after the end of suction was under water and the pump catch its prime without trouble.

There is no question but what, if the crib had been properly sheeted on the outside, and provided with a proper cutting edge, that the work would have proceeded with much less delay. The use of a clam-shell bucket for digging out the material would have been much better, had one been available, as not nearly so much sand and gravel would have come under the cutting edge as was sucked under by the pump, or as would have been sucked under had a hydraulic elevator been employed. The piles were driven in the crib about 50 feet below the cutting edge by the use of a 4200-pound hammer, and by the aid of two jets (Fig. 132). The average number of piles driven per day was only four, and could not be materially increased in the compact material encountered. About 15 feet of concrete was poured under water through a *trémie*, as soon as the sand between the piles had been cleaned out down to the bottom of the crib by using the sand-pump. As soon as this concrete was set the crib was pumped out by the centrifugal pump under a 28-foot head, and additional bracing placed as the water was pumped down, so that the timber was supported horizontally about every 6 or 8 feet, and vertically about 3 feet between bracing at the bottom, but increasing in the vertical distance towards the top. The piling was then sawed off and the rest of the concrete put in in the dry. There were a number of small leaks between the timbers of the crib which were stopped by driving shingles into the cracks. The crib had already been calked with two threads of oakum, and, had the tar paper and outside

sheeting been added, it would doubtless have been perfectly tight. After it had been pumped out once, very little additional pumping was required to keep it dry. After the crib was sunk, and before concreting, it was fastened down to guide-piles which had been originally driven to locate the crib when it was landed, by means of caps across the top of the crib bolted to the piles. These caps had been in use throughout the sinking, and wedges driven underneath them to keep the crib down in case any accident happened. This was a very fortunate precaution, as, one night during the sinking, while the author was standing on the side of the crib, one of the outside

FIG. 132.—JETTING DEEP PILES, LULU ISLAND PIERS.

sand-boxes broke loose with a loud report, and, had it not been for this fastening, the entire crib would have been wrecked. As it was, no serious damage was done except for the need of rebuilding and refilling the sand-box. During the progress of the work the crib hung up on one corner, and divers were employed to go down and free it. It was supposed a log had been encountered and would have to be cut in two. The divers failed to find any log, and after they had given up the attempt to free the crib it was discovered that it was hung up by wedging on one of the guide piles, which had run in toward the crib in driving. The crib was wedged off of this and no further obstruction encountered.

CHAPTER X

OPEN DREDGED CAISSONS OF TIMBER

The further use of open dredged caissons or cribs combined with piling for very deep foundations is well shown in some piers recently constructed by the author at Tacoma, Wash. The new Eleventh Street bridge over the City Waterway, an arm of Puget Sound in Commencement Bay at Tacoma, consists of an approach from the bluff on the city side, made up of an abutment 97 feet 6 inches long, and a steel approach 475 feet long; a fixed span between piers No. 1 and No. 2, 190 feet center to center of piers; a lift span 221 feet center to center of piers; and a second fixed span 190 feet center to center of piers. The bridge carries a 50-foot paved roadway, and two 10-foot sidewalks. The piers are shown on the profile in Fig. 133. Wash borings made by the Consulting Engineers showed hardpan at pier No. 1 at a depth of about 80 feet below high water; at pier No. 2 at a depth of about 105 feet, at pier No. 3 at a depth of about 118 feet, and at an unknown depth at pier 4. The borings at pier No. 1, as shown in the profile, disclosed first, a layer of mud and shells, then, blue clay overlying blue clay and sand, and beneath that coarse sand and gravel overlying the hardpan. At the other piers the borings showed principally fine sand, and towards the bottom of the piling on pier No. 3 some clay. These borings were made by the wash-boring process, and turned out to be no more reliable than is usual with such other examinations of like character. In sinking pier No. 1, hardpan and cemented gravel were encountered at about the point where blue clay is shown; at pier No. 2 the material turned out to be about as shown, and at pier No. 3 the material proved to be of sand and mud down to a depth of 160 feet below high water, where it was supposed that hardpan was reached; at pier No. 4 the material was fine sand and mud to an unknown depth.

The caissons (Fig. 134) were 18 feet 4 inches×72 feet 2 inches for piers No. 1 and No. 4, and 21 feet×81 feet 6 inches for piers No.

2 and No. 3. The elevation of high tide is +18, and of low tide elevation −2. The caissons were to be sunk in each instance to elevation −34, or about 52 feet below high tide. The caissons were constructed on the north shore on launching ways (Fig. 135), with an inclination of $1\frac{1}{2}$ inches per foot, which is practically twice that called for in the usual rules for launching ways for small vessels, which is based on a coefficient of friction of 0.04 for the ways well coated with skid grease. The ordinary rules for laying down launching ways for small vessels being from $\frac{3}{4}$ inch to 1 inch per foot, for average-sized vessels $\frac{5}{8}$ inch to $\frac{3}{4}$ inch per foot, and for largest vessels $\frac{1}{2}$ inch to $\frac{5}{8}$ inch per foot. Should it be required to pull the caissons back on to the ways, the force required may be determined from the following formula:

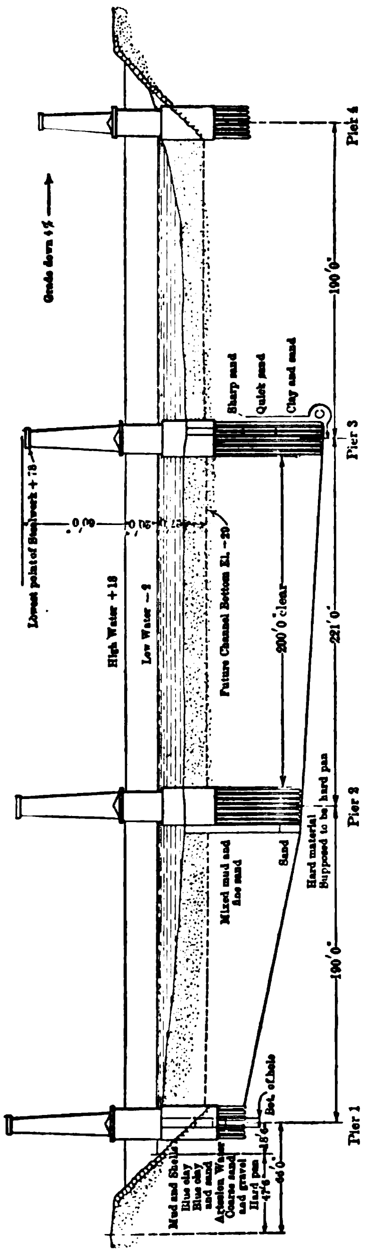


Fig. 133.—Tacoma, Wn., Profile of Piers and Soundings.

$$F = \text{Weight } (\sin a + u \cos a)$$

F being the force required, a the angle of inclination of the ways, and u the coefficient of friction, which may run up to several times the value given above, where the ways are not well coated with grease. The coefficient of friction for timber ways only soapy is 0.2, and for dry timber 0.5 maximum. The launching of the caissons was carried out without any trouble, Fig. 136 showing one of the caissons striking the water, and Fig. 137 showing the same caisson listed over so that the interior

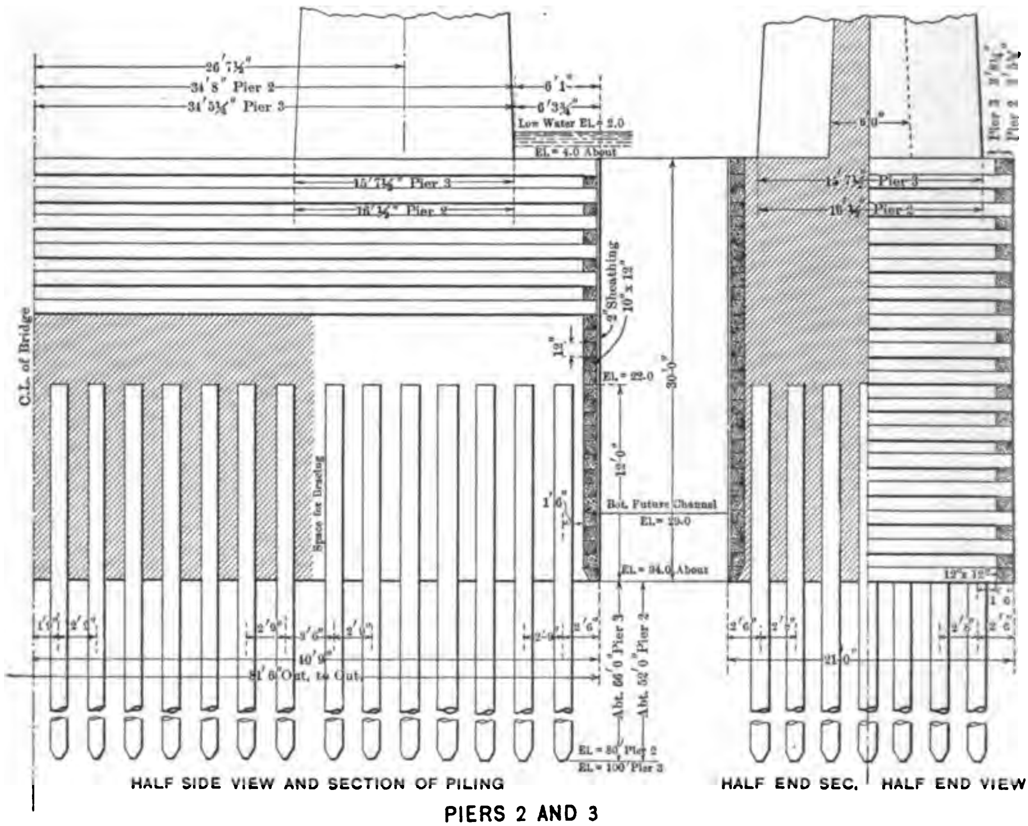
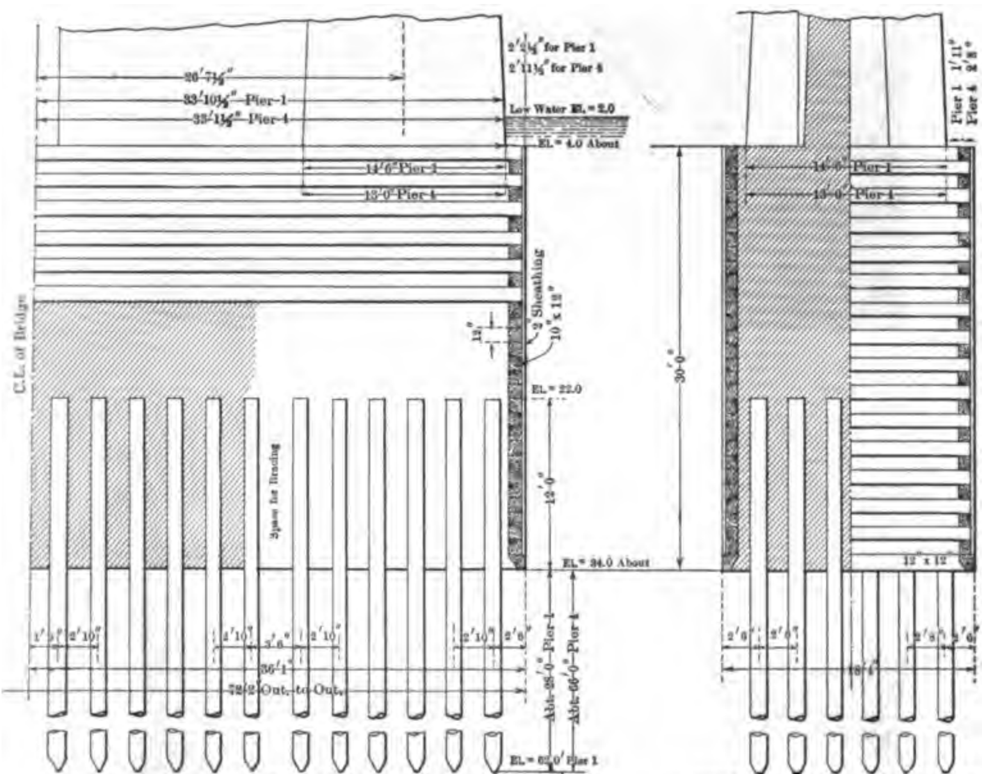


FIG. 134.—CAISSONS, ELEVENTH STREET, TACOMA. (To face page 195.)



HALF SIDE VIEW AND SECTION OF PILING

HALF END SEC. HALF END VIEW

℄ PIERS 1 AND 4

bracing can be seen. Additional data on launching will be found in Chapters XI and XII.

FIG. 135.—CAISSON ON WAYS, TACOMA.

It was impossible to place more bracing in the caissons during the sinking than was shown by the plans, on account of the number of piles that had been driven in each one, 159 in No. 1, 198 in No. 2, 206 in No.

FIG. 136.—LAUNCHING CAISSON, TACOMA, WN.

3, and 144 in No. 4. Steel cutting edges were used on piers No. 1 and No. 2, where it was expected that the caissons would be sunk with

the hydraulic elevators, but as the cutting edges were found to be of no service in excavating the caissons with clam-shell buckets, they were omitted on the other two cribs, and this had the effect of allowing caissons to list badly, without the weight of the metal to anchor them down.

The timber used in building the cribs was 10×12 inches surfaced on all four sides, with two $\frac{1}{2}\times 2\frac{1}{2}$ calking edges run on at the mill. The timbers were laid up on edge, log-house fashion at the corners, breaking joints at the brace frames, and the bottom course chamfered off, as shown into a rough cutting edge. Each timber was drift-bolted to the one below by $\frac{7}{8}\times 22$ inch drift-bolts every three

FIG. 137.—INTERIOR OF CAISSON, TACOMA, WN.

feet. The seams were calked on shore, up to above flotation line, with two threads of oakum, and the 2×12 inch surfaced sheathing spiked on to break joints with the ones to be put on above after the cribs were launched. The timber work was built up to a height of 22 feet before launching, and after launching twelve courses were added, calked and sheeted before the cribs were located. In addition to the bracing already mentioned, there were added inside each corner 12×12-inch vertical timbers, to brace the corner framing.

The method employed in sinking the caissons was to drive guide-piles at one side and both ends of each pier site, and the caissons were then towed to the exact location, and guide-piles driven on the other side. The plan originally was to build up the upper part of the cribs or coffer-dams in plank sections, as shown in Fig. 138, but as this necessitated the taking off of the sand-boxes or weight used

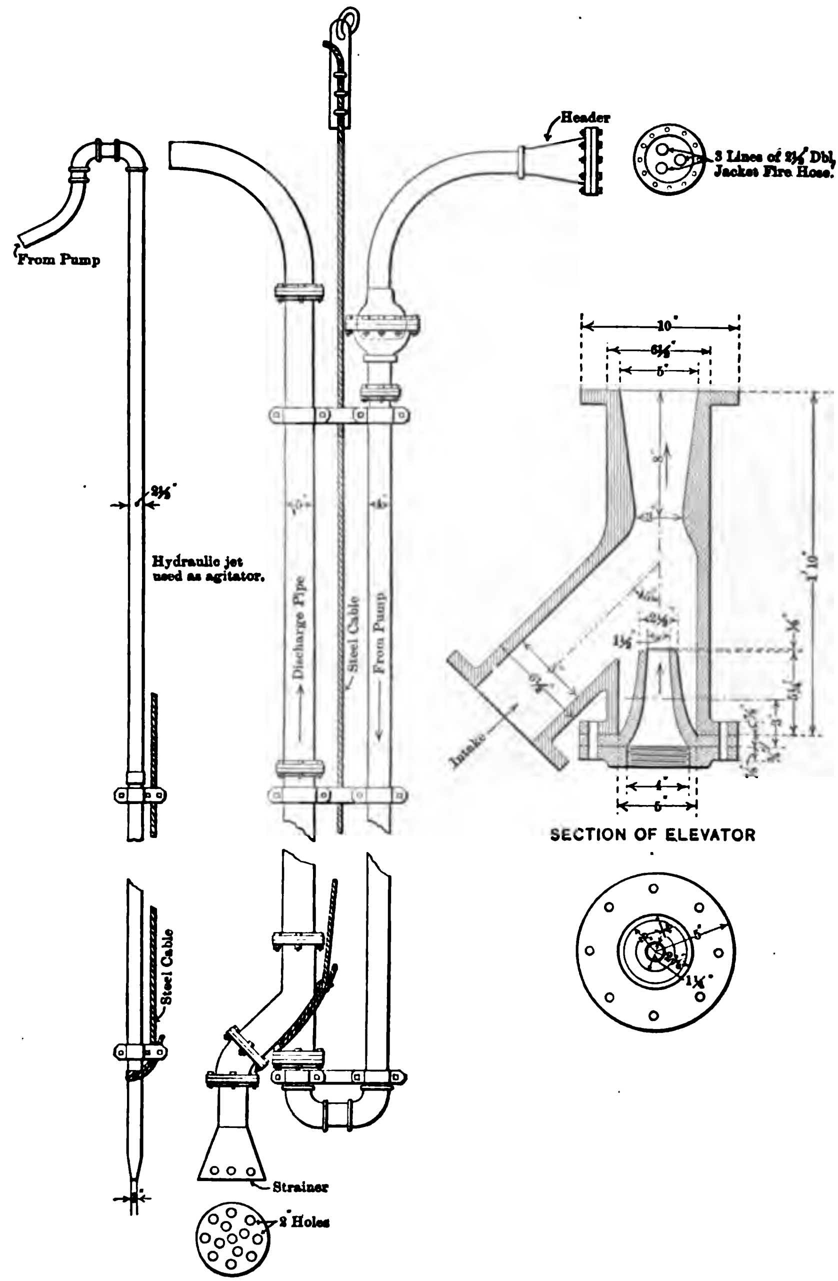


FIG. 139.—HYDRAULIC ELEVATORS, TACOMA, WN.

for sinking, and the rebuilding of the sand-boxes a second time, this was abandoned for piers Nos. 2, 3, and 4, and the coffer-dams built up to about elevation +12 of square timbers, drift-bolted on the same as the caissons themselves. This required only one section of the plank coffer-dam to be bolted on, to reach above high tide. Excavation was begun on pier No. 1 with sand-pumps, or hydraulic elevators, Fig. 139, but as it was found that the water jets would not loosen up the heavy material properly, derrick scows were provided, and Owen clam-shell buckets used for the greater portion of the depth of pier No. 1, and for the entire depth of all

FIG. 140.—SAND BOX LOADING, TACOMA PIERS.

the other piers. The cribs were weighted down by placing timbers crosswise over the tops, and building sand-boxes of old plank plank (Fig. 140) and filling them with sand up to about 400 tons to overcome flotation and friction, as the material was dug out of the cribs by the clam-shell buckets. This indicated a skin friction for the depth in the bottom of from 220 to 300 pounds per square foot. It was necessary to place the timbers far enough apart on the top of the cribs so as to allow plenty of room for the operation of the clam-shell buckets. The excavated material was mostly loaded on to scows and dumped in deep water at a distance from the bridge

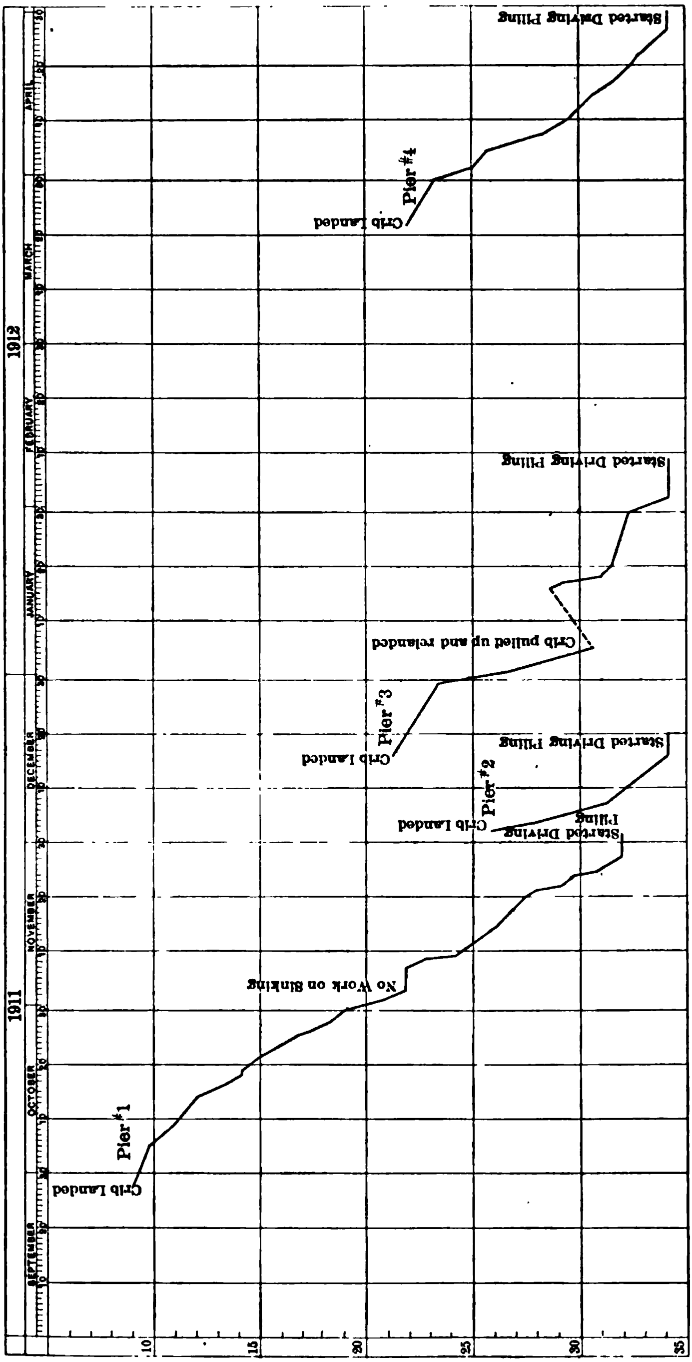


FIG. 141.—SINKING DIAGRAM, TACOMA CAISSONS.

of about a mile. By examining the sinking diagram, Fig. 141, it will be seen that the sinking proceeded very slowly for pier No. 1 and it was finally stopped about elevation −32. As the material had proven to be so hard, piling were ordered long enough to reach up to about mean tide, where they could be driven by a 4200-pound drop-hammer without jetting, to about the depth of −62, or 28 feet below the cutting edge, as originally planned.

FIG. 142.—DRIVING 125-FT. PILES.

Before locating the crib for piers Nos. 2, 3, and 4, the bottom was dredged out to within about 10 or 12 feet of the bottom of the crib as finally landed, or as deep as it was possible to do economically, with the material running from a considerable distance into the hole. The only trouble encountered in sinking the cribs was on pier No. 1, where a blow-in occurred when the crib was pumped out, by one corner of the crib giving way, either through too long spacing of the braces for the great head of over 40 feet, or through

defective timbers, it being impossible to determine which, as the timbers were broken into kindling wood. This occurred just after the men had come out of the crib to change shift, and no one was injured. On pier No. 3 the crib capsized during sinking, and had to be recalked, the sand-boxes replaced and refilled. This was due to insufficient fastening down, as all of the cribs were fastened down to the guide-piles after they had reached the full depth, so that the sand-boxes could be removed to allow of the piles being driven without any interference. The foundation piles for pier No. 2 were about 90 feet in length, and they were driven with a 4200-pound hammer, the most effective shaped one the author ever used, Fig. 142*a*, aided by two 2½-inch jets supplied from a 12×7×10 duplex pump on a power scow alongside. After the piles had been driven as deep as possible by the direct contact of the hammer, a follower of about 50 feet in length was used to follow them down to the required depth, or to about elevation −98. The piles for pier No. 3 were followed down to an elevation of about −139, or practically 160 feet below high water, using 125-foot piles and the 50-foot follower. (Fig. 142.) Handling jet pipes of between 160 and 170 feet in length made the operation very tedious, and often not more than two piles were driven in an eight-hour shift, with a maximum on this pier of four piles driven in any one shift. The long piles for pier No. 4 were driven as many as 9 in one shift.

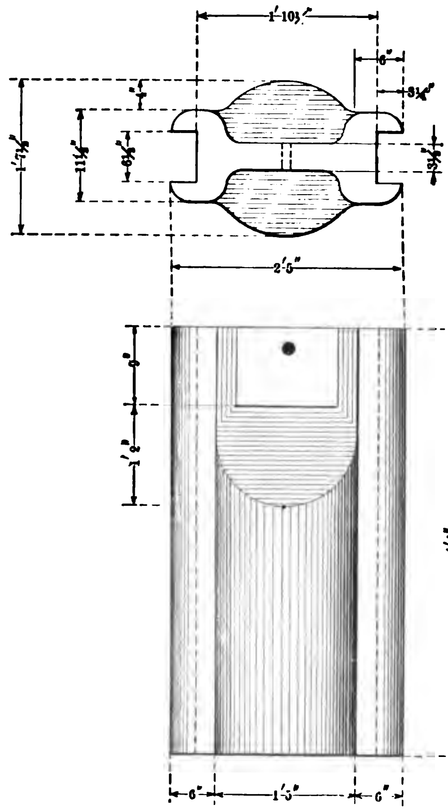


FIG. 142*a*. DROP HAMMER, 4200 LBS.

The piles, 125 feet long, were required by the Consulting

Engineers to be perfectly straight, which was a very severe requirement, and they were only obtained at a price of 18 cents per lineal foot. The process of driving them consisted first in shaping up the heads to fit the cast-steel follower caps; the jets were run down into the bottom to the full depth, a pile dropped in and with the 4200-pound hammer operating and the jets kept going up and down the pile to lubricate it, the pile would be driven down to the water surface, wherever that was at the time. Then the hammer and follower cap were raised, the 50-foot follower inserted, the follower cap again lowered, and the pile driven and jetted to full depth. The depth reached was supposed to land on the hardpan, as determined by running the jets down, although it was very uncertain as to whether it was hardpan or very compact sand or gravel. These piles had surface enough to carry over 85 tons by skin friction. This would indicate that the piles had about twice the penetration necessary. As pier No. 4 had only one end of the fixed span to carry, 90-foot piles were used, reaching down to approximately elevation −100. None of the piles in piers Nos. 2, 3, and 4 were cut off after driving, as they were driven with the follower cap, the head of the pile being framed to fit the cap, and the heads found to be left in first-class condition by cutting off some samples by the aid of a diver (Fig. 143), and the diver making an examination of the balance. The reason for this was that the cribs had been designed so light that it was not found safe to pump them out to the depth that had been intended originally, and they were finally pumped out only down to about elevation −10, to the top of the concrete put in by the *trémie*, or a maximum head of about 28 feet. Additional bracing was put in as they were pumped down, so as to make them entirely safe. The experience gained on the work would indicate that the cribs should have been made of not less than 14-inch timbers, and the cribs large enough in both dimensions to have allowed additional bracing during sinking, at least to the extent of one additional set longitudinally and transversely. In fact, as the piles were figured to carry a maximum load of 50 tons, the area of the cribs should have been very much larger, in order to allow a greater number of piling and to reduce the load on the piles down to a maximum of not over 35 tons per pile.

In every case after the piles had been driven, it was found that the material had swelled up between the piles, filling in the caissons from 4 feet in some instances, to about 9 feet in caisson No. 3 where the piles were driven to such a great depth. This material was removed by hydraulic elevators (Figs. 129 and 139) working in

among the piles, at the rate of about 0.2 foot per hour, each hydraulic elevator having attached to it a water jet to stir up the material in pier No. 1, but a separate jet in piers Nos. 2, 3, and 4. After the material was all cleaned out the concrete filling was deposited under water through a 10-inch pipe *trémie* (Fig. 144) up to about elevation −10. After allowing the concrete to set for about one week, the cribs were then pumped out and braced as the water was lowered,

FIG. 143.—HEAD OF PILE AFTER DRIVING WITH FOLLOWER.

and the balance of the concrete deposited practically in the dry. Where any water was encountered from leakage, the concrete was poured so as to force the water ahead, and not run any risk of washing out the cement. The concrete specified to be deposited under water was in the proportion of one of cement, two of sand, and three of clean washed gravel. This was mixed on the concrete scow alongside by No. 3 Ransome mixer, driven by a 15-H.P. electric motor. The mixer dumped into a bottom-dump one-yard concrete bucket which was handled from the mixer to the *trémie* hopper by a steam stiff-leg derrick on the same scow.

This *trémie*, as previously stated, was made up of 10-inch flange-jointed steel pipe, so that it was water tight, and it was not necessary to keep the concrete above the water level, as is necessary with

FIG. 144.—DEPOSITING CONCRETE THROUGH TRÉMIE, TACOMA.

riveted-pipe *trémies* which are not calked. The bottom of the *trémie* was kept buried in the concrete several feet, and as the concrete was deposited by moving the *trémie* around and lifting it up

slightly, it would not come in contact with the water, and practically no washing of the cement occurred; so that when the caissons were pumped out and the concrete examined, it was found to be of practically uniform texture, and, in some cases, more so than that deposited in the dry. After an experience of a quarter of a century in handling concrete, the author would not hesitate to state that under competent foremen as good—or better—concrete can be placed under water by a *trémie* of this sort as can be gotten by deposition in the dry.

The concrete that was placed in the dry was in proportion of 1–3–5, the size of the gravel being 2½ inches and under. The

FIG. 145.

cement was specified to weigh 380 pounds per barrel, and each sack was considered to contain 1 cubic foot. These proportions were not strictly adhered to, as it was found necessary to vary them as conditions changed during the progress of the work. One of the completed piers is shown in Fig. 145.

The plant used on the work consisted of the following floating equipment and tools:

The steam tugboat *Miami*, 65 feet long by 14 feet 8 inches beam.

The gasoline tug *Scioto* 30 feet long by 9 feet beam.

Six sand and gravel scows about 24×80×7 feet deep.

One derrick barge $30 \times 90 \times 8$ feet with $1\frac{1}{4}$-yard Owen bucket. (Fig. 146.)

One derrick barge $28 \times 80 \times 8$ feet with one-yard Owen bucket.

One power scow $24 \times 72 \times 4.5$ feet with duplex pumps and boilers.

One floating pile-driver $20 \times 60 \times 4$ feet with 65-foot leads. (Fig. 147.)

One coal scow $18 \times 37 \times 3.4$ feet.

One cement scow $21 \times 36 \times 3.5$ feet.

Two material scows about $25 \times 75 \times 7$ feet.

One skid driver with 65-foot leads.

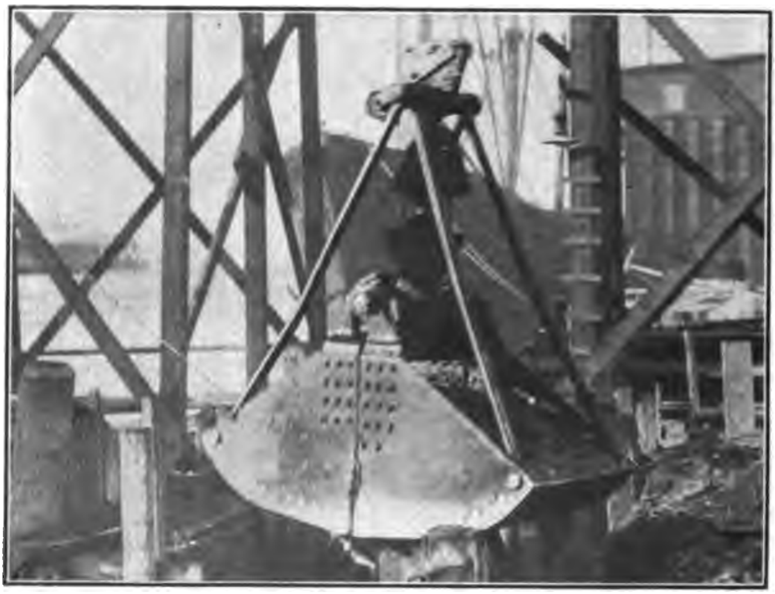


FIG. 146.—OWEN BUCKET ON TACOMA PIERS.

One Ransome concrete mixer, 30 cubic feet capacity, 15-H.P. motor.

The hoist engines were used as follows:

Double-drum, double 7×12 engine with 120 H.P. Scotch boiler on large derrick scow.

One double-drum, double 7×10 hoist engine, on small derrick scow.

One double-drum, double 7×10 hoist engine on the concrete scow.

One double-drum, double 7×10 engine, on caisson yard derrick.

One double-drum, double 7×10 hoist engine on skid driver.

One 30-H.P. locomotive boiler, and two locomotive air-compressors in caisson yard.

The power scow carried:
One 80-H.P. vertical boiler.
One 40-H.P. vertical boiler.
One 30-H.P. locomotive boiler.
One $12\times7\times10$ duplex pump.
One $7\frac{1}{2}\times4\frac{1}{2}\times10$ duplex pump.
One $7\times4\frac{1}{2}\times10$ duplex pump.
The other large items of the plant were:
One 3-inch hydraulic elevator.
One 4-inch hydraulic elevator.

FIG. 147.—FLOATING DRIVER, TACOMA.

One Sullivan air compressor $9\times10\times12$, with 40-H.P. vertical boiler, on west bank adjacent to machine shop and blacksmith shop.

One Ransome concrete mixer for shore work, operated by 10-H.P. electric motor, 14 cubic feet capacity.

One 6-inch sand-pump, with direct-connected 8×7 steam engine.

One 6-inch sand-pump, with 9×9 vertical belted engine.

Also a full equipment of small plant and tools.

The work was in general charge of a Superintendent, under whom acted the Assistant Superintendent, who was also the Resident Engineer for the contractors. The office work was carried on under

the direction of the Resident Engineer, who also had charge of the timekeepers, material man, and the ordering of all material and supplies.

Reporting direct to the Superintendent were the caisson foremen, dredge foremen, pile-driver foremen, carpenter foremen, concrete foremen, tugboat captains, yard foremen, and bridge carpenter foremen and steel foremen.

The figures given for skin friction on the sides of the caissons during the sinking, 220 to 300 pounds per square foot, are probably lower than the maximum that occurred, inasmuch as it was impossible to load the caissons as heavily as desired to accelerate the sinking, on account of the thin walls and the lack of sufficient bracing.

The figures given by various authorities for the skin friction during sinking of piers are so various as to be of little service in carrying out any particular work.

The friction on the Roebling East River bridge caissons was found to be 900 pounds per square foot. The record does not state the exact depth or the character of the material in which this occurred, but the air pressure would indicate the depth to be between 40 and 50 feet.

The friction on some brick and cement cylinders in the River Clyde is stated to be about 1300 pounds per square foot, although the depth is not stated. The friction on cast-iron cylinders for the Chittrivatri bridge is given at 280 to 450 pounds per square foot through 33 feet of sand, 10 feet of clay, 7 feet of clay and sand, and clay and boulders.

Gaudard states that the friction on cast-iron cylinders will reach as high as 2 or 3 tons per square foot at small depths, and from 4 to 5 tons at a depth of from 20 to 30 feet!

From these figures it will be seen the published data is very unreliable, and it will not pay to take up space by giving further figures of this sort. Where the values are to be used as part of the carrying capacity, they cannot safely be taken in excess of those given in table herewith:

Where, however, the values are desired as a guide to the necessary loading for sinking caissons, they may be taken at a value of not less than twice that given in the table, and will very often reach to four or five times the values given.

The fact that it will be very easy in any important case to make experiments, either separately or during the progress of the work, to determine the exact amount of the friction, makes it obligatory for the engineer to carefully examine the case in hand. The suction

outside or the uplift of the air inside very often enter largely into the values that are arrived at, and great care must be taken to avoid being misled in this way. On the other hand, with pneumatic caissons, the released air coming from under the cutting edge will very greatly reduce friction on the sides of the pneumatic caissons, and a high enough result cannot be obtained unless careful consideration is given to the exact conditions and circumstances at the time the experiment is made or the figures prepared.

TABLE XXI.—FRICTION ON CAISSONS.

POUNDS PER SQUARE FOOT.

Material.	Depth below Ground Surface in Feet.				
	25	50	75	100	125
Silt or soft mud	50	75	100	125	150
Stiff mud	100	125	150	175	200
Packed sand	250	300	350	400	450
Clay and ordinary gravel	300	350	400	450	500
Heavy packed gravel	350	400	450	500	550
Hardpan and cemented gravel	450	500	550	600	650

Most probable value from experiments, provided original material is in close contact with caisson. Softer material may have replaced it, and corresponding value must be used.

The great difficulty of properly loading a caisson with only single walls unless they are made extra heavy, leads to the conclusion that except for very small caissons, and for those to be sunk only a short depth, they should be made with double walls similar to the Northern Pacific pivot pier, Fig. 128, so that they can be loaded with the concrete filling that will remain a permanent part of the pier, and thus avoid the great expense and trouble of temporary loading. It is true that with double-wall cribs they must be sunk to a depth where the need of piling to prevent scour will be avoided, and in this case, of course, the caissons will have to be large enough in plan to keep within proper limits of bearing for the character of the bottom on which they are to be stopped, taking into account, of course, friction and buoyancy.

CHAPTER XI

TIMBER PNEUMATIC CAISSONS

THE construction of foundations by the pneumatic or compressed-air process is the most reliable method for important foundations where the work is difficult or where the piers are to be carried down to great depths, with a limit of about 110 feet below the surface of the water. The caissons for this work in the United States are almost invariably constructed of timber, as well as in other countries where timber is plenty, and one of the most recent examples of this kind are the piers for the double-track bridge for the Northern Pacific Railway over the Columbia River at Vancouver, Wn.

FIG. 148.—VANCOUVER, WN. BRIDGE. GENERAL VIEW.

The work was carried out under the general charge of W. L. Darling, Chief Engineer of the Northern Pacific Railway, from the designs of Ralph Modjeski, Consulting Bridge Engineer for the road. The construction was carried out under the immediate charge of B. L. Crosby, Member of the American Society of Civil Engineers, Resident Manager, who had an efficient corps of superintendents and foremen.

The bridge is, as stated, a double-track structure to carry the Spokane, Portland & Seattle Railway over the Columbia River into Portland, Ore. The structure as shown in Figs. 148 and 149

is 2806 feet 10 inches in length, consisting of the following spans, beginning at the north end. One through fixed span 189 feet 6 inches in length; one through draw 446 feet 10 inches in length; one through fixed span 374 feet 8 inches in length; six through fixed spans 269 feet 0 inches long; and one through fixed span 162 feet 0 inches long.

The foundation for this bridge consists of a north abutment built in an open excavation on the gravel bottom. Pier No. 1 was founded on a pneumatic caisson 26.27 feet below low water; pier No. 2 was constructed on a caisson sunk some years before to a depth of 60 feet below low water; pier No. 3 on a pneumatic caisson 70.19 feet below low water; and piers Nos. 4 to 10 inclusive founded on pneumatic caissons from 70 to 80 feet below low water. The type of these piers and caissons is shown in Figs. 150, 151 and 152. The last pier,

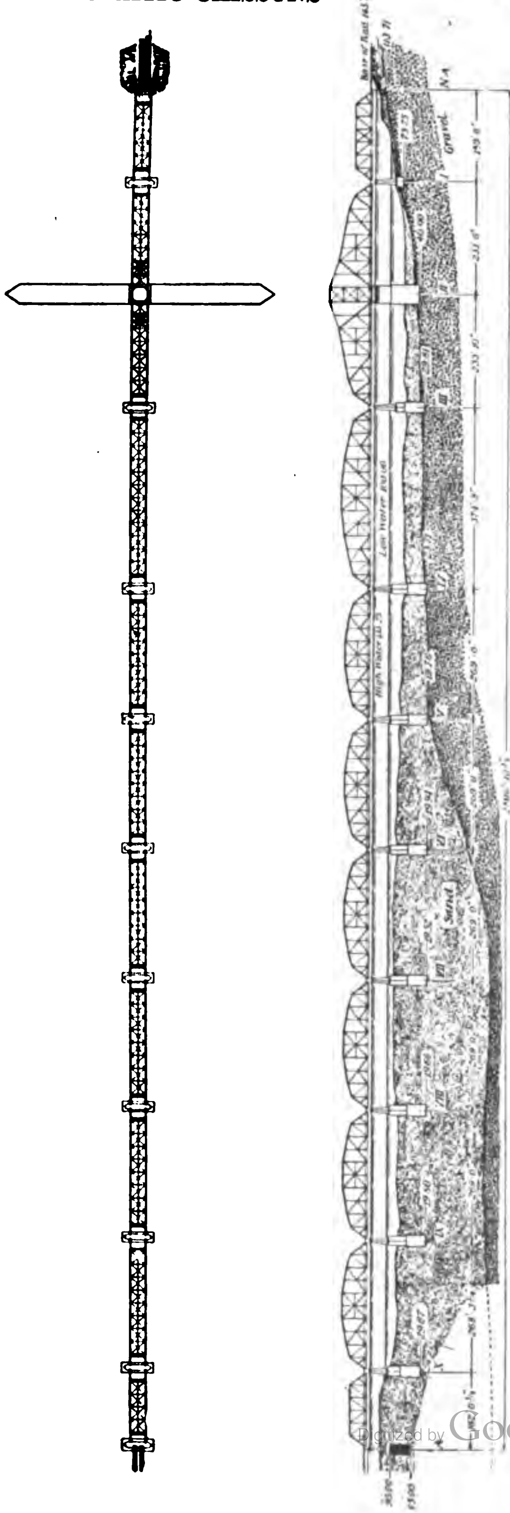
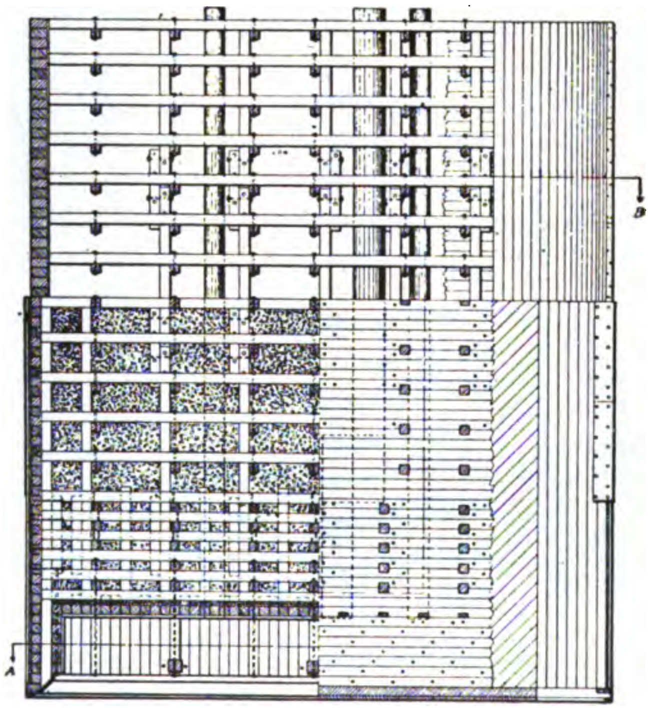


FIG. 149.—VANCOUVER BRIDGE. GENERAL ELEVATION AND PLAN.

which is also common to the viaduct over Hayden Island, is founded on piles. The caissons were 21 feet in width for all piers except pier No. 4, which was 23 feet in width; had a uniform length of 59 feet and height of 40 feet. They were constructed on launching ways (Fig. 153), on the north or Washington shore, the ways having an inclination of approximately 1½ inches to the foot. The construction of these and the cradle are plainly shown in the photograph.

Type of Caissons III to X inclusive.



Sectional Side Elevation C.D.

FIG. 150.—CAISSON FOR VANCOUVER, WN., PIERS.

After being completed to the height of 20 feet, they were launched sidewise and towed to the various locations and landed in the ordinary manner. It will be noted that the coffer-dam portion of the caissons is curved on the up-stream end in order to reduce the scouring action of the current. The tops of the cribs were mostly fixed at an elevation 5 feet below low water, with the exception of piers Nos. 1 and 3, where the tops were 12½ feet below low water, in order that

more channel room would be provided. The square caisson of pier No. 2 had been originally sunk in about 1890 by the pneumatic process down to the gravel, and the launching ways used for this were found to be in such excellent condition that they were utilized for the caissons of the present bridge, which was begun early in 1906.

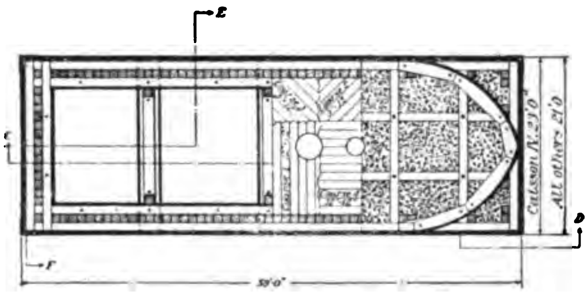


FIG. 150a.—PLAN OF VANCOUVER, WN., CAISSONS.

All the caissons from 1 to 4 inclusive were carried down to the gravel, as shown on the profile in Fig. 149, while those from 6 to 10 inclusive were carried down to a depth believed to be safe from any future scouring action.

The working chamber of the caissons was 8 feet 6 inches in height, and braced transversely with three-brace frames. The roof of the

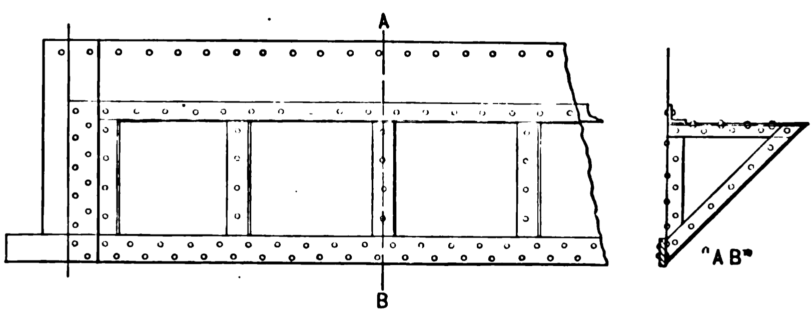


FIG. 150b.—MORISON CUTTING EDGE.

chamber was built up of 12×12 S.4S. timber with calking edges, for the first layer, then a second course of 3×12 S.4S. with calking seam, and a third course of 3×12 S.4S. with calking seam. The side and end walls were laid up of 12×12 S.4S. with calking seam and fastened together with 1×34-inch drift-bolts every 2 feet. The

outside walls of the caissons were of 12×12 S.4S. with calking seams, drift-bolted the same as the inside walls, and were sheeted with two layers of 3×12 S.4S. plank, with calking seam, the inside layer placed diagonally and both layers spiked on with $\frac{3}{8}$×8-inch boat-spikes. The inside of the working chamber was sheeted with $2\frac{1}{2}$×12×S.4S. plank, with calking seam and spiked on with $\frac{3}{8}$×8-inch boat-spikes. All the seams were calked with two threads of oakum. The coffer-dams were built up to a height of 40 feet of 12×12 walls and cribbing as shown in Fig. 150, and above this with 12×12 cribbing, the curved ends being cut from 12×18. The outside had calking seams and was calked with two threads of oakum the same as the lower portion.

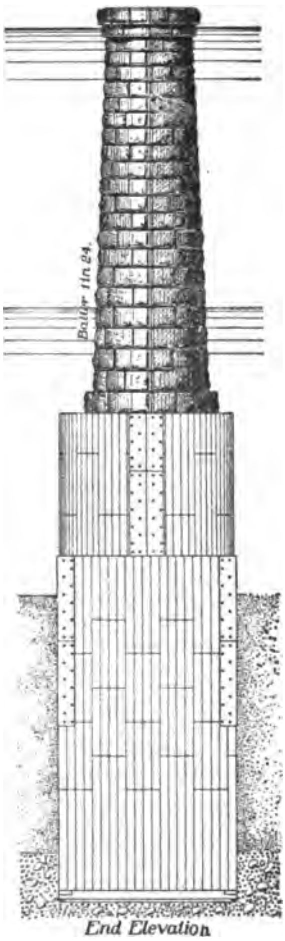


Fig. 151.—End Elevation, Vancouver Piers.

The corners of the upper half of the rectangular cribs on the up-stream side were protected with steel-angle plates $\frac{1}{4}$×48-inch, drift-bolted on with $\frac{1}{2}$×7-inch countersunk drifts. The angle or nose of the curved up-stream end of the coffer-dam was also protected against abrasion by steel angle-plates $\frac{1}{4}$×48-inch, drift-bolted on. No metal cutting edge was used, but a 4×8 oak shoe was drift-bolted flat on to the bottom timber of the caisson.

This was deemed sufficient for the material to be encountered, although for many of the timber caissons used in other places in the United States, elaborate metal cutting edges similar to those on the Forth bridge were employed. The details of those used on the Union Pacific bridge at Omaha, by the late George S. Morison, Consulting Engineer, are shown in Fig. 150*b*.

When the caissons of the Columbia River bridge had been launched and located, they were further built up, filled with concrete enough to land them, and concrete was deposited during sinking, as was found necessary, to overcome the flotation and friction.

The sinking of the piers was a comparatively easy piece of work,

the lowest rate of sinking being in gravel and boulders, an example of which is the bottom 5 feet of pier No. 3 (Fig. 154), which occupied, from May 12th to June 7th, twenty-six days, or an average of only about $2\frac{1}{4}$ inches per day. Piers Nos. 6 and 9 are examples of rapid sinking, the maximum being about 5 feet per day.

The material lock, Figs. 155 and 155*a*, is shown in enough detail to require no explanation. The air-lock for the workmen constructed of the Morison type, was used on four of the caissons; in the others, sections of the 36-inch main shaft were converted into air-locks by

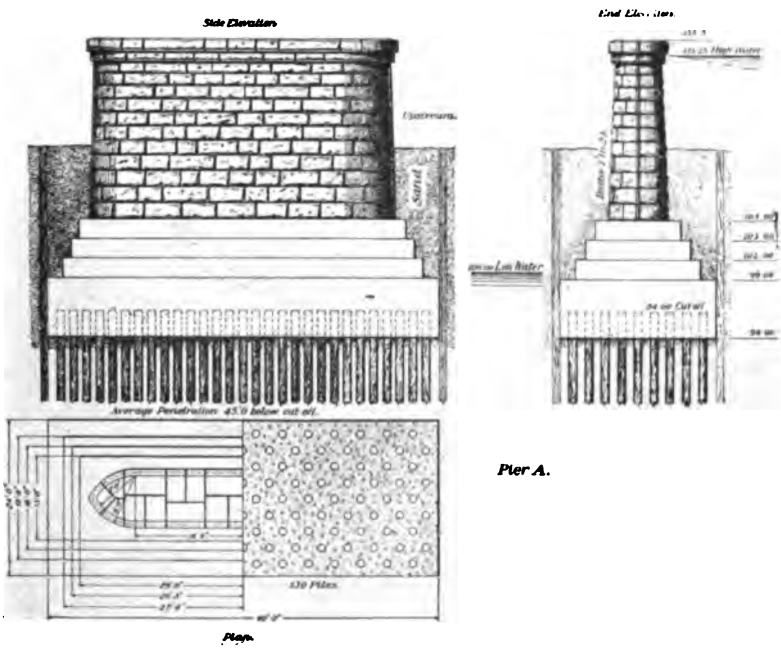


FIG. 152.—PIER A, VANCOUVER BRIDGE.

the use of diaphragms, containing doors, placed between adjoining sections of a shaft.

The material was removed by the wet blow-out process, excepting where gravel and boulders were encountered which had to be hoisted out. The Morison sand-pump, used on many of the Mississippi and Missouri River bridges, is shown in Fig. 157, and should be made of cast steel.

The plant on this work consisted of the following boats, scows, and machinery:

The steam tug *Edith*, of 74 gross tons, length 78.7 feet, beam 17.7 feet, depth 9.3 feet.

Stern-wheel tugboat *Metlako*, 198 gross tons, 122 net tons, length 109 feet, beam 24.4 feet, depth 4.8 feet.

Masonry derrick barge, 30×90×5 feet, cost $1900, carrying a 10-ton derrick with $8\frac{3}{4}\times 10$ Lidgerwood engine, and Smith concrete mixer, No. 5 (old No.).

Caisson derrick barge, 30×80×6 feet, carrying a 5-ton derrick with 7×12 Mundy engine, and small Chicago pneumatic air compressor.

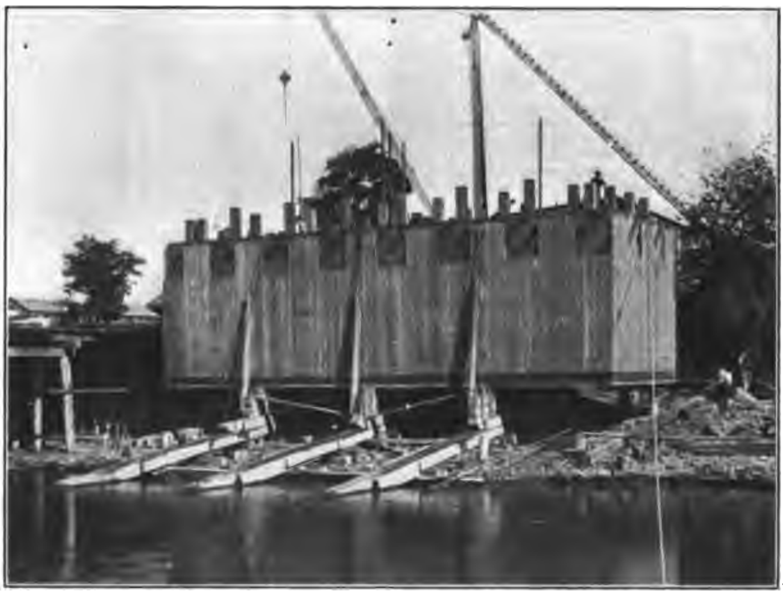
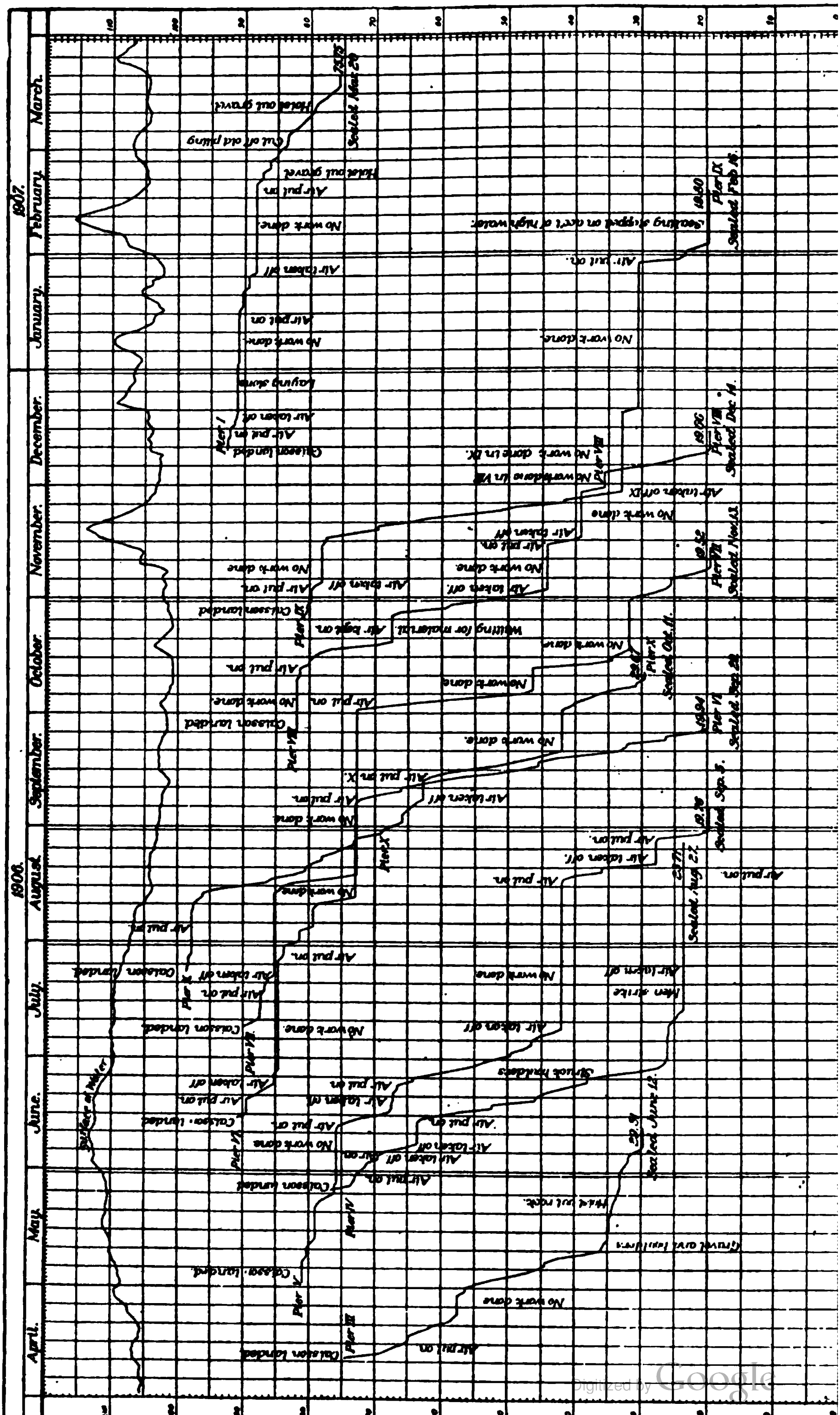


FIG. 153.—CAISSON ON LAUNCHING WAYS, VANCOUVER.

Concrete derrick barge, 30×90×5 feet, cost $1900, carrying a $7\frac{1}{4}\times 10$, and a 7×12 Mundy engine; two derricks, each 5 tons, and a Smith No. 5 (old No.) concrete mixer. (Chapter XXVI.) (Fig. 156.)

Pile-driver on scow, 22×70×4 feet, cost $1045, carrying an $8\frac{1}{4}\times 10$ Am. Hoist & Derrick Co. engine, and a No. 2 Vulcan steam-hammer.

Power barge, 32×124×6 feet, cost $2850; with $20\times 24\frac{1}{4}\times 24$ Ingersoll-Sargent straight-line, Class A. L. P. compressor; a $12\times 12\frac{1}{4}\times 14$ Ingersoll-Sargent straight-line, Class A. H. P.; a 100-light (16 c.p.) electric-light plant, 6×6 Hill automatic engine; pressure


1906.
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pump, 8×6×12; duplex boiler-feed, 6×4×6; three 80-H.P. fire-box boilers.

Power barge, 32×124×6 feet, cost $2850; with 20×24 high-pressure Norwalk compressor; a 150-light (16 c.p.) electric-light plant, with Westinghouse 18-H.P. engine; duplex feed-pump, 6×4×6; duplex pump, 7×4½×10; two old locomotive boilers.

FIG. 155.—MATERIAL AIR-LOCKS, VANCOUVER, WN.

Five barges, 20×75, depth 5 feet; cost of three $1140 each, cost of two $1189.40 each.

Eight barges, 24×92, depth 5 feet; cost of each $1520.

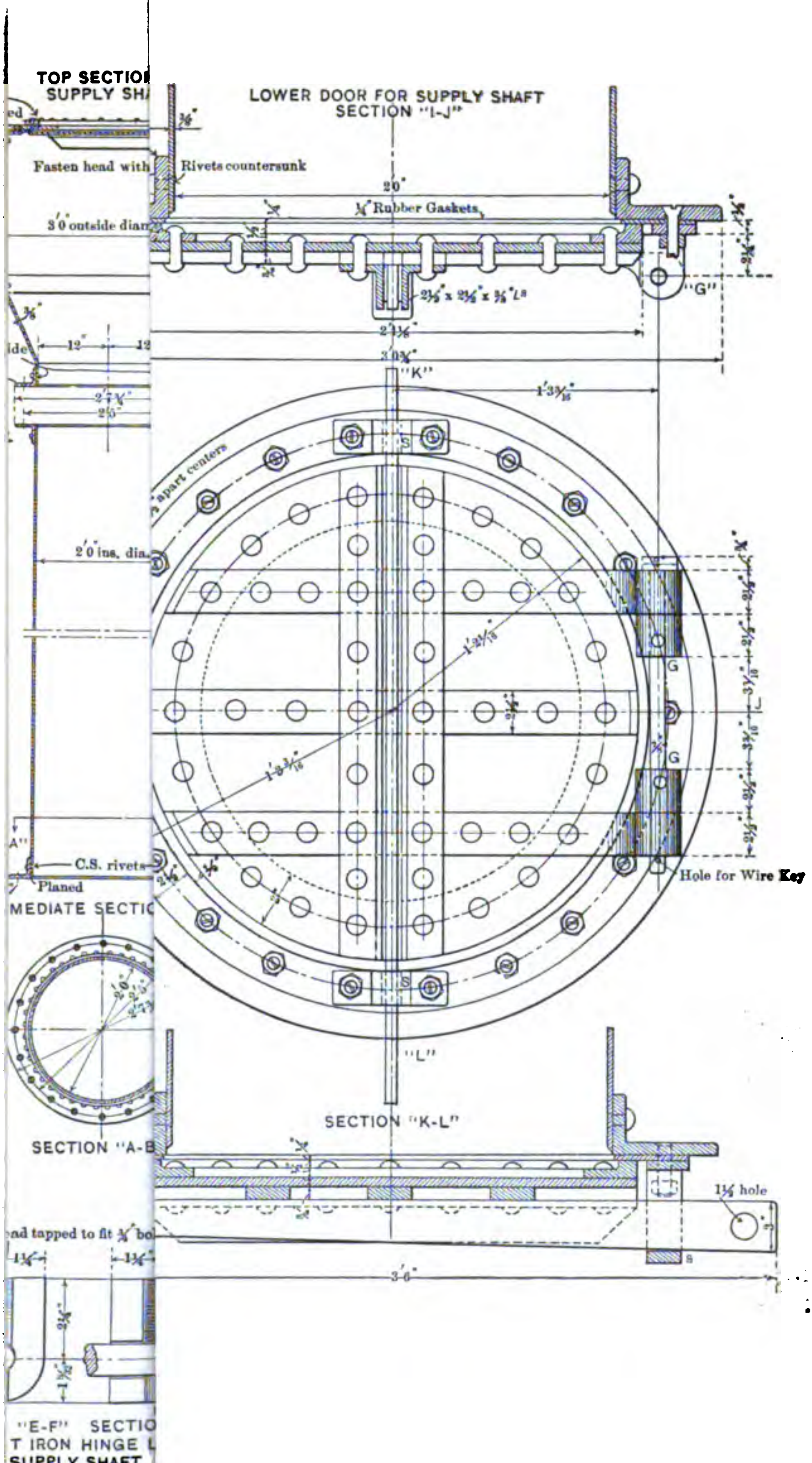
One barge, 26×92, depth 5 feet; cost of barge $1706.48.

One barge, 30×80×6 feet, cost second-hand $1200.

Shore Plant:

10-ton stone yard traveler.

5-ton derrick for caissons, with 8¼×10 Am. H. & D. Co. engine.



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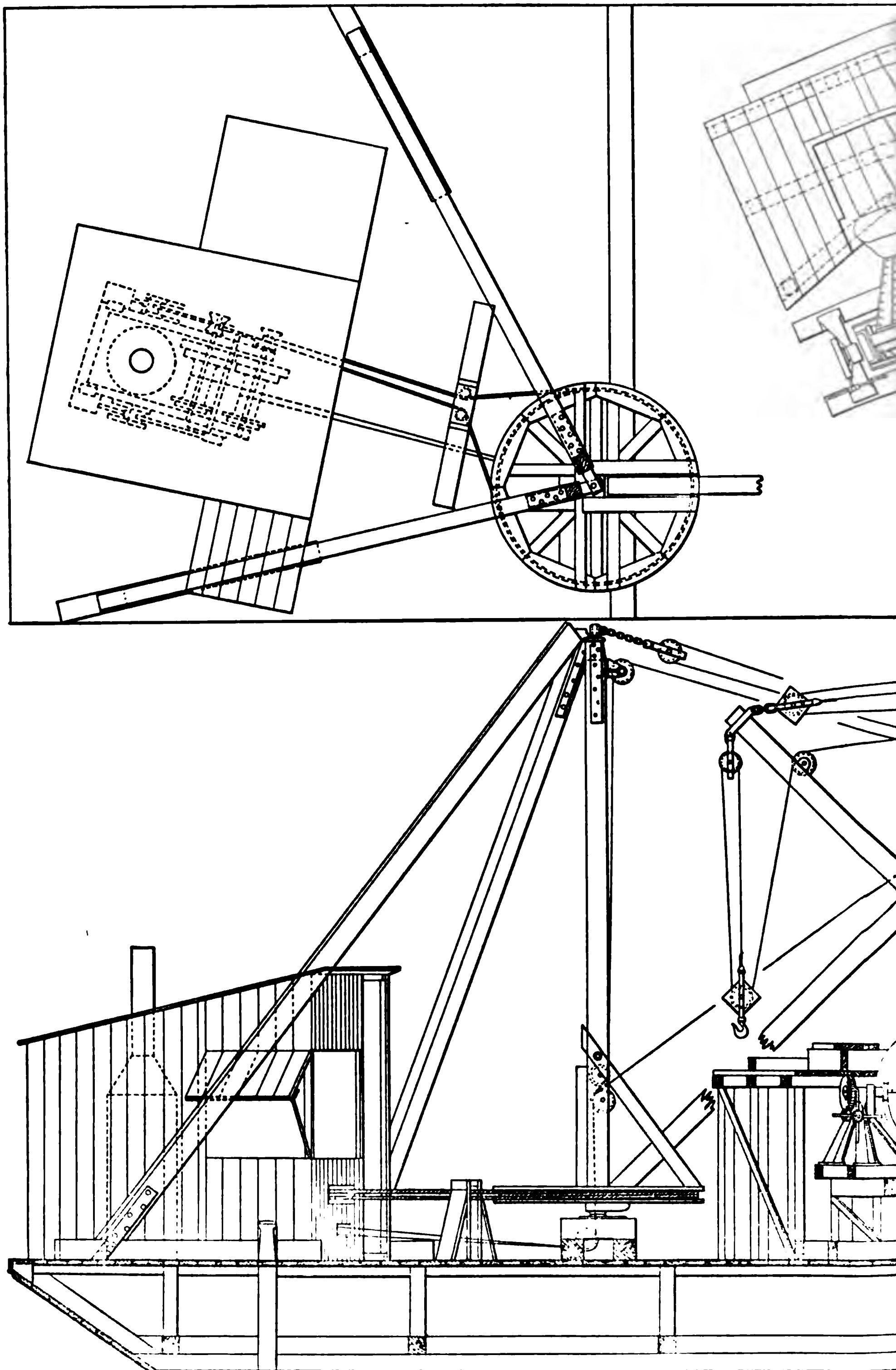
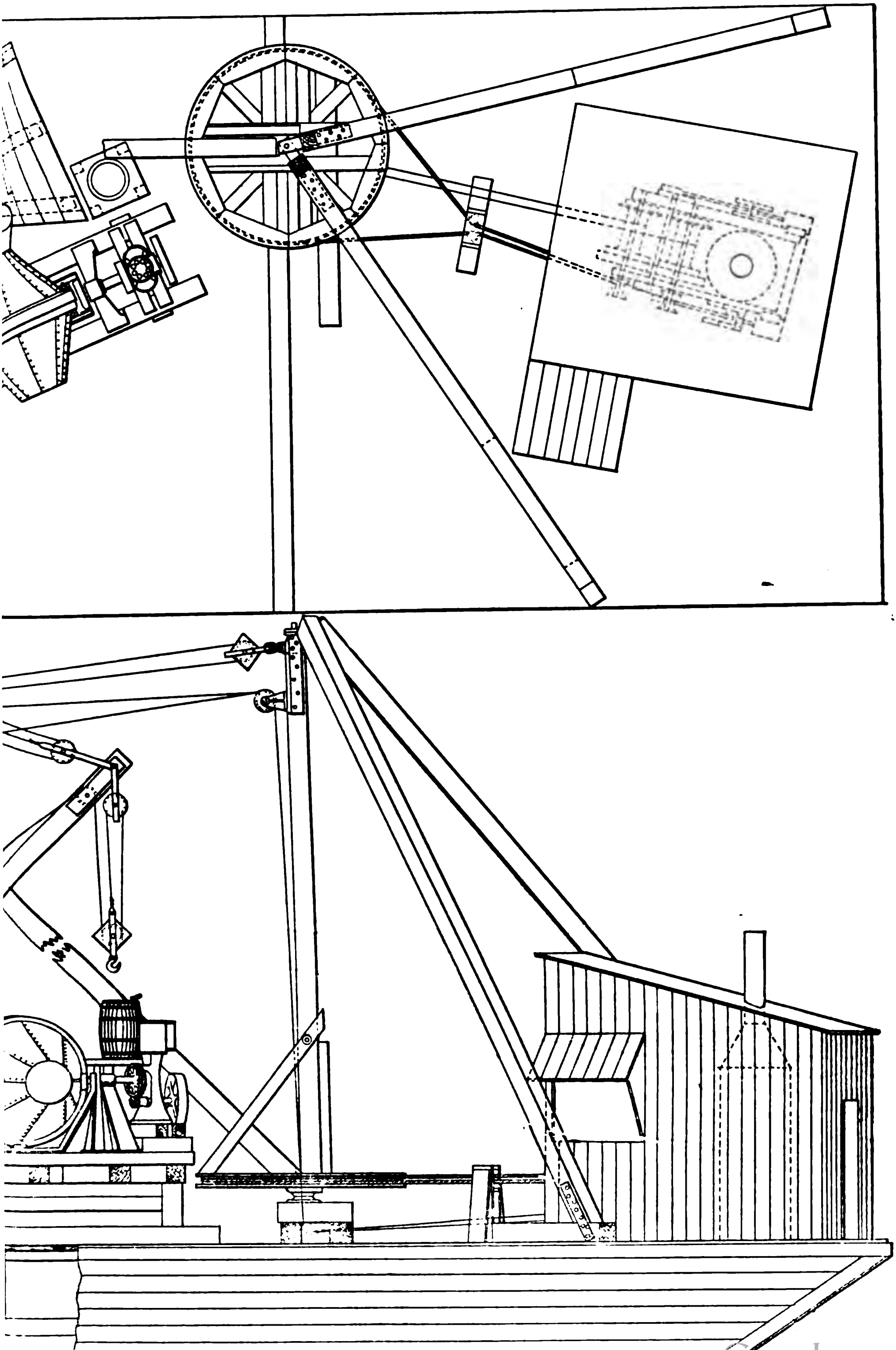


FIG. 156.—FLOATING CON



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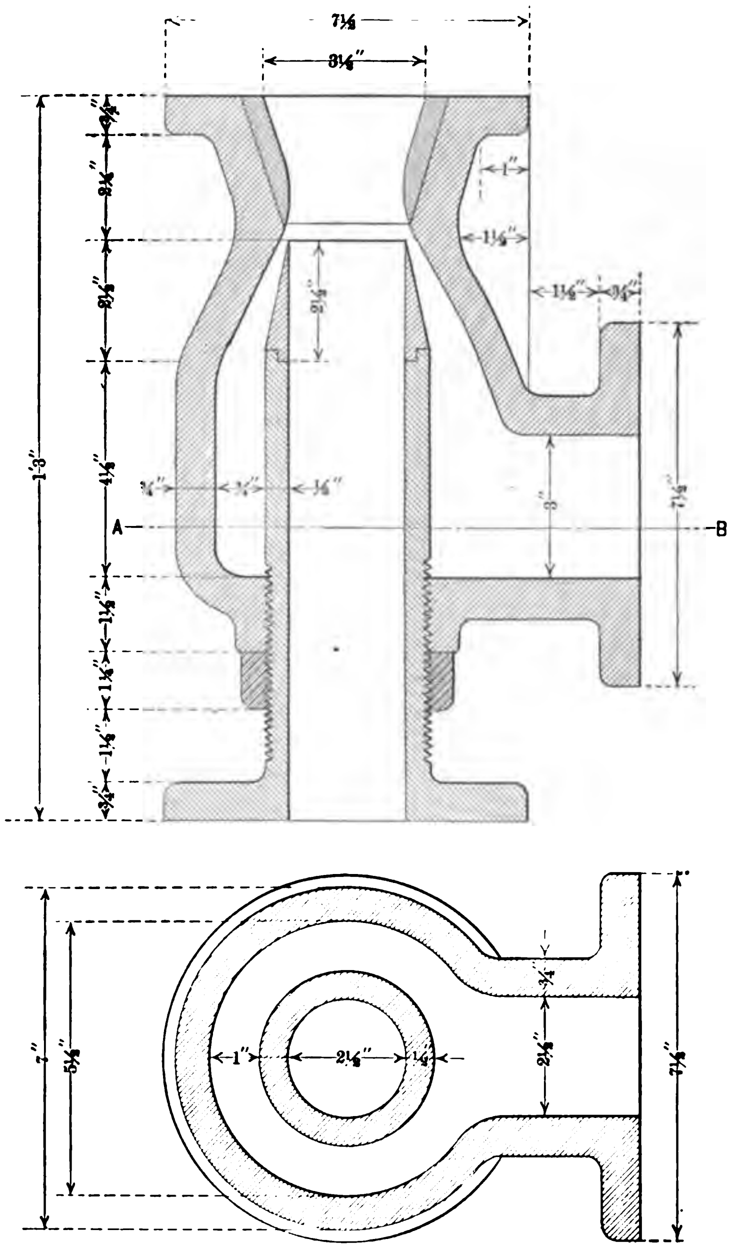


SECTION "A-B"

FIG. 157.—MORISON SAND-PUMP.

22-H.P. Fairbanks-Morse gasoline compressor.

10-ton derrick on deck, with $8\frac{1}{4}\times 10$ Lidgerwood engine.

5-ton unloading derrick, with $7\frac{1}{2}\times 12$ Mundy engine, and 1-yard Hayward clam-shell.

5-ton portable derrick, with $7\frac{1}{4}\times 10$ Mundy engine, and a No. $2\frac{1}{2}$ Smith mixer. (Old No.)

Pile-driver hoist engine, Mundy, 7×12.

Steam hammer, Vulcan No. 2.

Pneumatic caissons cannot be considered as coming strictly under the head of ordinary foundations, although there are many cases where they could be employed much more cheaply than the methods which are finally adopted, besides assuring a first-class piece of work, where with some other method there may be more or less guesswork about the result obtained. In the construction of an ordinary highway bridge at Chillicothe, Ohio, in 1898, after designs by the author, it was decided by the engineer, A. W. Jones, to employ pneumatic caissons in founding the piers, as it was possible to let the contract for slightly less than $13 per cubic yard of caisson and crib, or not very much in excess of what any other type of pier would have cost. One of these piers is shown in detail in Fig. 158, carried to bed-rock, 30 feet below low water. The caisson of this pier was 7 feet 8 inches in width, 27 feet 8 inches long and 6 feet 2 inches high. The out-to-out dimensions were 12 feet 4 inches in width by 32 feet 4 inches in length. The sides and roof of the caisson were 2 feet in thickness, built up of 12×12 timbers, each course being fastened to the one below with drift-bolts 30 inches long and spaced 4 or 5 feet centers. The two courses on the sides and ends were firmly fastened together by two rows of $\frac{7}{8}$-inch bolts extending through both courses. The two bottom layers of timber were beveled off on the inside to form a 4-inch "cutting edge," this being protected by a 2×6 plank firmly spiked on, which was very satisfactory, as all the timber was of oak. The working chamber was divided into three equal "pockets" by three tie-beams dovetailed into the sides of the caisson. The roof was pierced by five holes; 4-inch holes near each end for the "blow-out" pipes and an 18- and a 20-inch hole in the center "pocket" for supply-pipe and air-shaft. Near the air-shaft was another 4-inch hole for the air-supply pipe. The air-shaft itself was formed of $\frac{1}{4}$-inch boiler iron thoroughly riveted and calked and fastened together with inside flanges bolted together with $\frac{3}{4}$-inch bolts. Supply-pipe was formed of $\frac{3}{16}$-inch plates, and was similar in construction to the air-shaft, except that the flanges were on the outside. The inside of the cais-

son from the "cutting edge" to the roof and all over the roof was calked carefully in each joint between the timbers and around the bolt-heads. Then the sheathing was put on and this was thoroughly calked (Fig. 159). The caissons were built on level ways on the river bank (Fig. 160), and when ready for launching these ways were

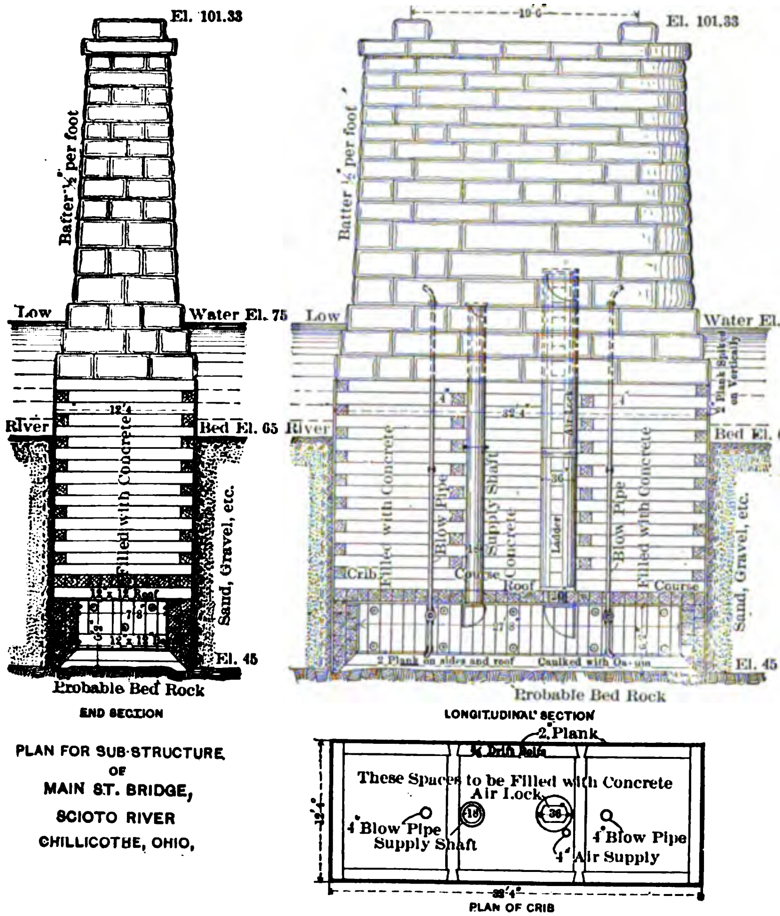


FIG. 158.—PNEUMATIC CAISSON, CHILLICOTHE, OHIO.

inclined by raising the inner end, and the launching was accomplished by starting the caissons with tackle until they slid off into the water.

On top of the caissons, cribs were built of 12×12 timbers with cross-ties in every other course and 30-inch drift-bolts every 4 or 5 feet. This was sheathed continuously from "cutting edge" to the top. After grouting the deck or roof of the caisson several feet of

concrete was put in until the pier became heavy enough to land on the bottom. The concrete was composed of 1 part of Portland cement, $2\frac{1}{2}$ parts of clean sand, and 5 parts of gravel, and the contractor was allowed to place large, clean bowlders in each layer.

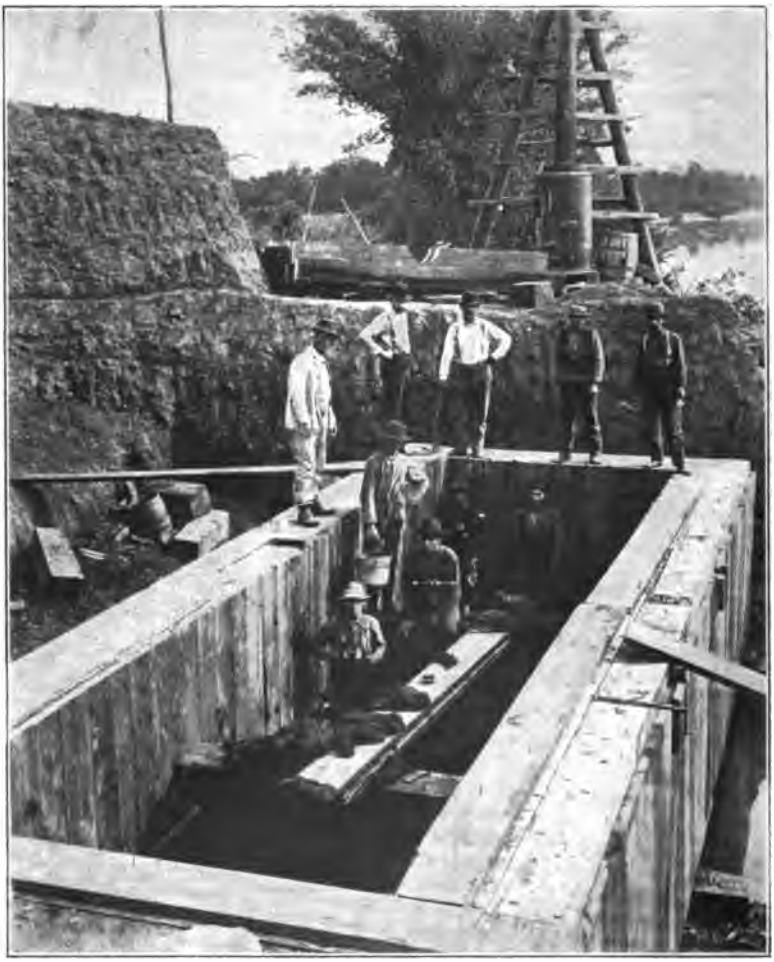


FIG. 159.—CALKING CAISSON NO. 2, CHILLICOTHE, OHIO.

The plant for sinking the piers consisted of a double compressor $14 \times 16 \times 18$, run by a 120-horse-power locomotive boiler. The air-receiver was 56 inches in diameter and 15 feet long, while the lighting plant consisted of a 115-volt dynamo of 9 amperes capacity run by

a vertical engine. In addition to this there were all the necessary pumps, hoisting-engines, derricks, and the like for carrying on the work. A 3-inch pipe was run from the receiver, along the river bed,

FIG. 160.—CAISSON NO. 1 ON WAYS, CHILLICOTHE, OHIO.

to each pier and connected to the supply-pipe by a flexible joint. The working chamber was lighted with three 16-c.p. electric lights, and one was placed in each section of the air-shaft. Work was begun

as soon as the air was turned on, by working two shifts of ten hours each. The "sand-hogs," or crews of men employed on the work, consisted of one foreman, two blowpipe feeders, four shovelers, an inside lock tender and an outside lock tender. The material was removed by means of ordinary sand-pumps and was discharged through "goosenecks" of cast iron outside the pier, these "goose-necks" being heavy enough to stand the wear for a considerable length of time. All rock too large to send out through the pipes was taken out through the air-shaft in sacks. In sinking the caisson all material was cleaned out level with the "cutting edge," then it was "ditched" by shoveling several inches of material from under the "cutting edge" and giving a "blow" by opening one of the valves and letting out a greater part of the air. Relieved of the lift of the compressed air, and with the weight of the concrete on top, the caisson settled down from 6 to 10 inches until the "cutting edge" was again on solid gravel. While the air was escaping, the working chamber filled with water and would frequently be two-thirds full when done "blowing." No particular trouble was had in sinking the piers except for caisson No. 1, where, at a depth of about 20 feet below the bed of the river, a layer of bowlders and several coping-stones from one of the old piers of the old bridge, which was being replaced, were encountered. The bed-rock under this pier was reached at 43 feet below water and consisted of black shale and limestone in alternating layers. After the caisson was landed the concreting of the air-chamber was begun.

To concrete the air-chamber a 2-inch pipe connection was made from the air-pipe to a point just below the top flange of the supply-pipe. This pipe had a valve, No. 1, near the air-pipe, and another valve, No. 2, on a T between valve No. 1 and the supply-shaft. A top door opening down was then put on the supply-shaft and the fastening taken off the lower door. A batch of concrete was shoveled into the supply-pipe, the upper door pulled up, valve No. 2 closed and No. 1 opened. This put compressed air in the supply-shaft, and allowed the lower door to equalize open and the concrete fell into the working chamber, where it was rammed under the bevel edges and over the bottom by the "sand-hogs." The lower door was then shut, valve No. 1 closed, and No. 2 opened, allowing the compressed air in the supply-shaft to equalize out and the top door to open. The process was repeated until the chamber was filled with concrete and barely a narrow passageway for one man left between the lower doors of the supply-shaft and air-shaft. The lower doors were then shut, and the last man equalized out. A regular air pressure

was then kept on for twelve or eighteen hours, until the concrete set. The air was then turned off, and the shafts filled with concrete as quickly as possible. As soon as the air pressure was taken off, the lower doors dropped and the space left in the working chamber also filled with concrete, which was run in from the shafts. This finished the foundation from bed-rock to above low water. The setting of the footing courses is shown in Fig. 161, and the completed piers and bridge in Fig. 161*a*.

FIG. 161.—SETTING FOOTING COURSES NO. 2.

The above account is taken practically verbatim from the report of A. W. Jones, the engineer on the work, to whom acknowledgment is made.

TABLE XXIII.—AIR PRESSURE LBS. PER SQUARE INCH.*
(See Chapter XIII.)

Depth.	Pressure.	Depth.	Pressure.	Depth.	Pressure.
30	13	100	43	170	74
40	17	110	48	180	78
50	21	120	52	190	82
60	26	130	56	200	87
70	30	140	61	210	91
80	34	150	65	220	95
90	39	160	69	230	100

* The rough rule used by divers and workmen is one half pound of air per vertical foot.

The subject of caisson disease, or bends, has been very fully treated in a paper by Henry Japp, Member of the American Society of Civil Engineers, in the Transactions of the American Society of Civil Engineers, Vol. LXV., who was Managing Engineer for S. Pearson & Sons, on the Pennsylvania Railway tunnels under East River at New York City. There were as many as 10 tunnel headings being worked at the same time, and a very rare opportunity was afforded of studying the question.

FIG. 161a.—SCIOTO RIVER BRIDGE, PIERS COMPLETED.

The old theory was that the nitrogen from the air was dissolved in the blood, but later studies show that it was due to a mechanical action by the dissolved air in the blood and tissues being liberated in the body in the form of bubbles, which expanded and tore the tissues, injured the spinal cord, brought about pressure on the brain, and frothed up the blood, slowing up or stopping the circulation and heart action. The most satisfactory cure has been found to be slow decompression, and a medical air-lock similar to Fig. 163 is, or should be, employed on all compressed-air work where the pressure exceeds 29 pounds per square inch, so that those suffering from bends can be placed in a medical lock under pressure, and by slow

decompression relieved or practically cured. To effect this decompression, and to properly ventilate the medical lock, the decompression valve, Fig. 164, is used. It was found, however, that if the men were not allowed to come out of the compressed air too quickly, that the number of men suffering from bends was practically none

FIG. 162.—BLOWING ON PNEUMATIC CAISSON, CHILLICOTHE.

at all. The decompression valve for use on the air-locks (Fig. 165), required the men to take 15 minutes to give uniform decompression from 35 pounds to atmospheric pressure, the one for the medical lock requiring 1 hour for 35 pounds pressure. T. Kennard Thomson, Member of the American Society of Civil Engineers, in discussing

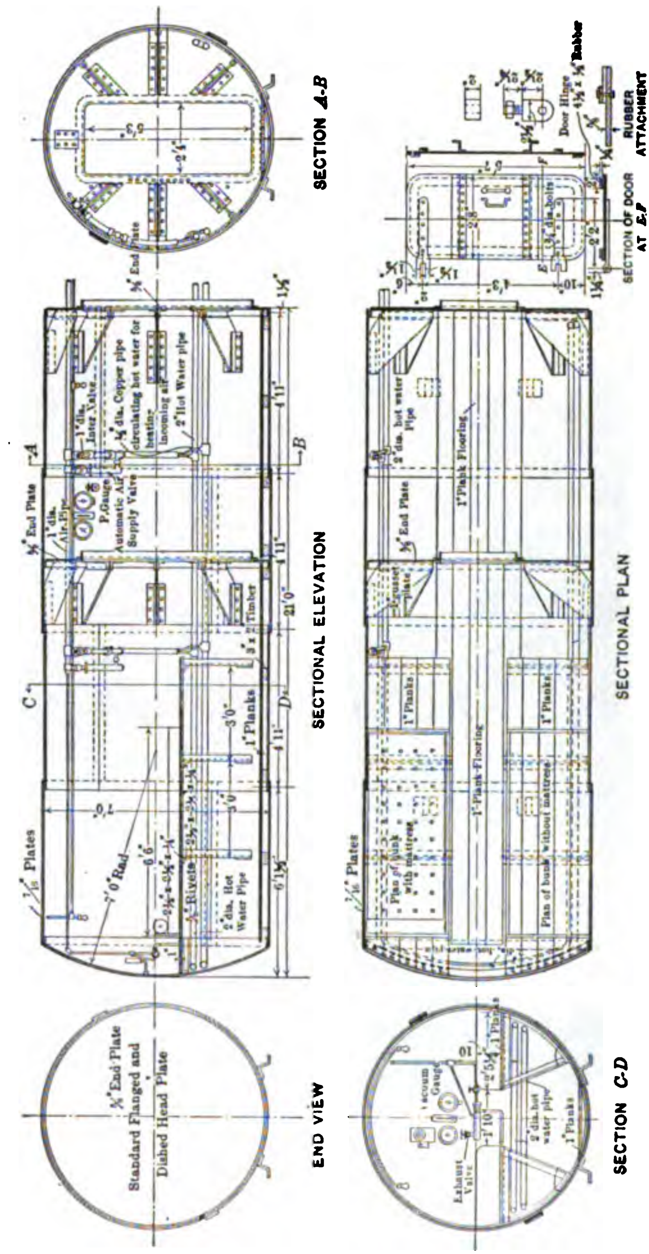


FIG. 163.—MEDICAL AIR-LOCK USED IN THE EAST RIVER TUNNELS.

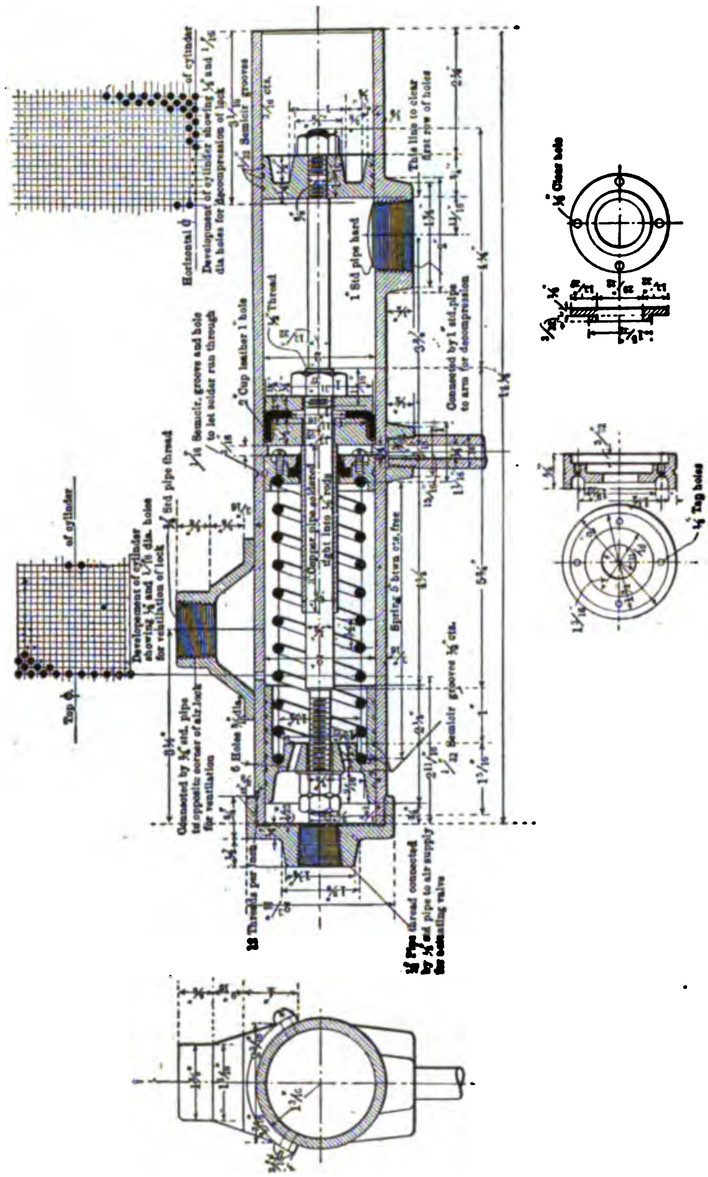


FIG. 164.—AUTOMATIC CONSTANT RATE DECOMPRESSION AND VENTILATING VALVE.

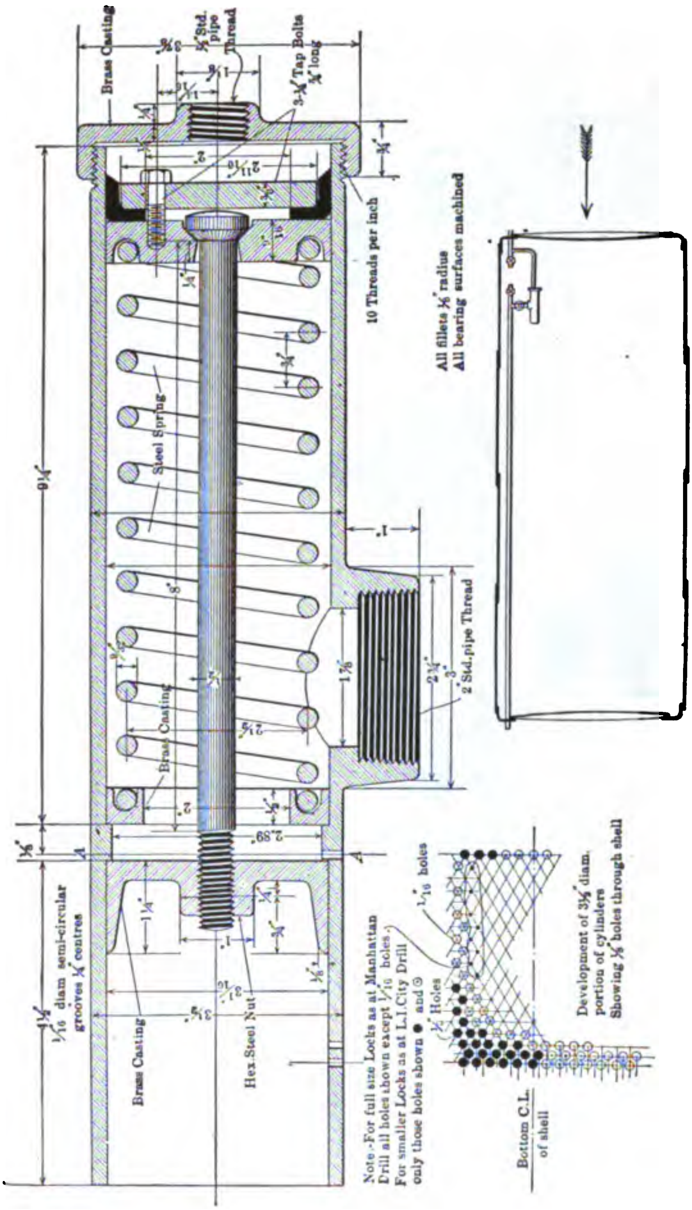


DIAGRAM OF CONNECTIONS

FIG. 165.—AUTOMATIC CONSTANT RATE DECOMPRESSION VALVE.

this paper states that the use of the electric battery will often cure the patient by the electric shocks after recompression has failed, although it is not nearly so reliable.

This has become a matter of so much moment that the State of New York has placed a law on its statute books on the subject which is given at length, although it does not entirely meet with the approval of many experienced in the use of compressed air.

Section 134*a*. HOURS OF LABOR. All work in the prosecution of which tunnels, caissons or other apparatus or means in which compressed air is employed or used shall be conducted subject to the following restrictions and regulations: When the air pressure in any compartment, caisson, tunnel or place in which men are employed is greater than normal and shall not exceed 21 pounds be square inch, no employee shall be permitted to work or shall remain therein more than eight hours in any 24 hours and shall only be permitted to work under such air pressure provided he shall, during said period, return to the open air for an interval of at least 30 consecutive minutes, which interval his employer shall provide for. When the air pressure in any compartment, caisson, tunnel or place in which men are employed is greater than normal and shall equal 22 pounds and does not exceed 30 pounds per square inch, no employee shall be permitted to work or remain therein more than six hours, such six hours to be divided into two periods of three hours each with an interval of at least one hour between each such period. When the air pressure in any such compartment, caisson, tunnel or place shall exceed 30 pounds and shall not equal 35 pounds per square inch, no employee shall be permitted to work or remain therein more than four hours, such four hours to be divided into two periods of two hours each, with an interval of at least two hours between each such period. When the air pressure in any such compartment, caisson, tunnel or place shall equal 35 pounds and shall not exceed 40 pounds per square inch, no such employee shall be permitted to work or remain therein more than three hours in any 24 hours, such three hours to be divided into periods of not more than one and one-half hours each, with an interval of at least three hours between each such period; when the air pressure in any such compartment, caisson, tunnel or place shall equal 40 pounds and shall not equal 45 pounds per square inch, no employee shall be permitted to work or remain therein more than two hours in any 24 hours, such two hours to be divided into periods of not more than one hour each, with an interval of at least four hours between each such period; when the air pressure in any such compartment, caisson,

tunnel or place shall equal 45 pounds per square inch and shall not exceed 50 pounds per square inch, no employee shall be permitted to work or remain there more than 90 minutes in any 24 hours and such 90 minutes to be divided into periods of 45 minutes each, with an interval of not less than five hours between each such period; no employee shall be permitted to work in any compartment, caisson, tunnel or place where the pressure shall exceed 50 pounds per square inch, except in case of emergency. No person employed in work in compressed air shall be permitted by his employer or by the person in charge of said work to pass from the place in which the work is being done to atmosphere of normal pressure, without passing through an intermediate lock or stage of decompression, which said decompression shall be, where the work is being done in tunnels, at the rate of three pounds every two minutes unless the pressure shall be over 36 pounds, in which event the decompression shall be at the rate of one pound per minute; and which said decompression shall be, where the work is being done in caissons, at the following rates:

Where the pressure is not over 10 pounds per square inch the time of decompression shall be one minute; when pressure is over 10 pounds, but does not exceed 15 pounds, the time of decompression shall be two minutes; when pressure is over 15 pounds, but does not exceed 20 pounds, the time of the decompression shall be five minutes; when pressure is over 20 pounds, but does not exceed 25 pounds, the time of decompression shall be 10 minutes; when pressure is over 25 pounds, but does not exceed 30 pounds, the time of decompression shall be 12 minutes; when pressure is over 30 pounds, but does not exceed 36 pounds, the time of decompression shall be 15 minutes; when pressure is over 36 pounds, but does not exceed 40 pounds, the time of decompression shall be 20 minutes; when pressure is over 40 pounds, but does not exceed 50 pounds, the time of decompression shall be 25 minutes.

All necessary instruments shall be attached to all caissons and air-locks showing the actual air pressure to which men employed therein are subjected, and which instruments shall be accessible to and in charge of a competent person who shall not be employed more than eight hours in any 24 hours.

Section 134*b*. Medical Attendance and Regulations. Any person or corporation carrying on any tunnel, caisson or other work in prosecution of which men are employed or permitted to work in compressed air, shall, while such men are so employed, also employ and keep in employment, one or more duly qualified persons to

act as medical officer or officers who shall be in attendance at all necessary times while such work is in progress, and whose duty it shall be to administer and strictly enforce the following:

(*a*) No person shall be permitted to work in compressed air until after he shall have been examined by such medical officer and reported by such officer to the person in charge thereof as found to be qualified, physically, to engage in such work.

(*b*) In the event of absence from work, by an employee for 10 or more successive days for any cause, he shall not resume work until he shall have been re-examined by the medical officer and his physical condition reported, as hitherto provided, to be such as to permit him to work in compressed air.

(*c*) No person known to be addicted to the excessive use of intoxicants shall be permitted to work in compressed air.

(*d*) No person not having previously worked in compressed air shall be permitted during the first 24 hours of his employment to work for longer than one-half a day period as provided in Section 134*a*; and after so working shall be re-examined and not permitted to work in a place where the pressure is in excess of 15 pounds unless his physical condition be reported by the medical officer, as heretofore provided, to be such as to qualify him for such work.

(*e*) After a person has been employed continuously in compressed air for a period of three months he shall be re-examined by the medical officer and he shall not be allowed, permitted or compelled to work until such examination has been made and he has been reported, as heretofore provided, as physically qualified to engage in compressed-air work.

(*f*) The said medical officer shall at all times keep a complete and full record of examinations made by him, which record shall contain dates on which examinations were made and a clear and full description of the person examined, his age and physical condition at the time examined, also the statement as to the time such person has been engaged in like employment.

(*g*) Properly heated, lighted and ventilated dressing rooms shall be provided for all employees in compressed air, which shall contain lockers and benches and shall be open and accessible to the men during the intermission between shifts. Such rooms shall be provided with baths, with hot- and cold-water service and a proper and sanitary toilet.

(*h*) A medical lock shall be established and maintained in connection with all work in compressed air when the maximum pressure exceeds 17 pounds as herein provided. Such lock shall be kept

properly heated, lighted and ventilated and shall contain proper medical and surgical equipment. Such lock shall be in charge of a certified trained nurse selected by the medical officer, who shall be qualified to render temporary relief.

(*i*) Whenever in the prosecution of caisson work in which compressed air is employed, the working chamber is less than 10 feet in length and when such caissons are at any time suspended, or hung, while work is in progress, so that the bottom of the excavation is more than nine feet below the deck of the working chamber, a shield shall be erected in the working chamber for the protection of the workmen.

(*j*) Whenever in the prosecution of work in which compressed air is employed, a shaft is used, all such shafts shall be provided with a safe, proper and suitable ladder for its entire length.

(*k*) Wherever in the prosecution of work in tunnels, caissons or other apparatus or means in which compressed air is employed or used, lights other than electric lights are used, the said lights shall at all times be guarded.

(*l*) All passage ways in work, wherein compressed air is employed or used, shall be kept clear and properly lighted.

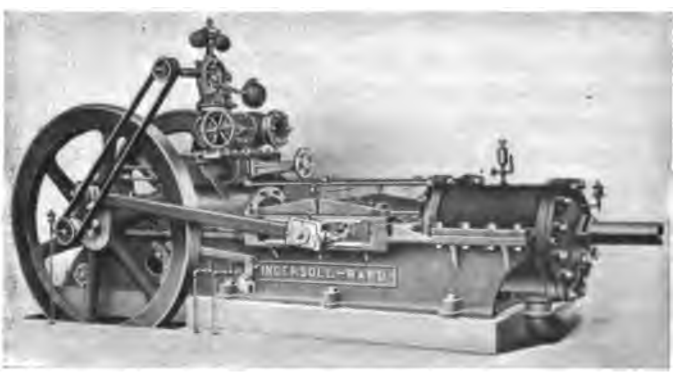


FIG. 165(*a*).—AIR COMPRESSOR FOR PNEUMATIC WORK.
Ingersoll-Rand Class "A-1" Compressor, Fitted with the "Air Ball" Governor; Standard on Larger Sizes.
For sizes and capacities see TABLE LXIX, APPENDIX XI.

CHAPTER XII

STEEL PNEUMATIC CAISSONS, FORTH BRIDGE*

THE metal coffer-dams used on two of the Inchgarvie piers of the Forth Bridge have been described in Chapter VIII, and this chapter will cover a description of the steel pneumatic caissons used on the work. The foundations and masonry for the bridge were given special attention by Sir John Fowler, as work of this character had been his especial forte during the many years of his wide practice on British engineering work in all parts of the world.

The bridge (Fig. 166) is slightly over one and one-half miles long, consisting of fifteen spans 168 feet long, two shore arms 689 feet 9 inches, two main spans 1710 feet, two towers 145 feet, one tower 260 feet, six masonry approach spans and two abutments.

The contract for the work was let on December 21, 1882, to Tancred, Arrol & Co., and they in turn let the contract for the pneumatic caissons to M. Coiseau, of Paris and Antwerp, who had large numbers of men experienced on the pneumatic work of the Antwerp Harbor construction.

The six piers sunk by compressed air were the southernmost ones at Inchgarvie and those at Queensferry. The caissons were built on launching ways (Fig. 167) on the sloping shore on the south side of the Estuary, the metal-work having been fabricated at the Dalmarnock Iron Works in Glasgow. The iron-work was assembled and riveted on the ways, which had a slope of about 1.1 ins. to 12 ins., and after launching the caissons were towed to their locations. (Fig. 168.) The material of the caissons was of the sizes shown in Figs. 169, 169*a* and 169*b*, the bottom courses or cutting edge being of $\frac{1}{2}$-inch plates and the upper courses of $\frac{3}{8}$-inch plates. The diameter of the steel shells was 70 feet for all six piers, the height of the working chamber 7 feet, and the total height of the caissons from 50 feet to 60 feet. The shells were stiffened with angle irons, and the inner and outer shells latticed with angle bracing. The roof of the

*The steel pneumatic caissons used on the Antwerp Quay Wall are fully described in Chapter XXIX, and should be carefully studied.

FIG. 166.—FORTH BRIDGE, SCOTLAND.

working chamber was supported by four lattice girders 18 feet in depth, as shown on Fig. 169*b*, and by thirteen plate girders of 4 feet depth at right angles to these deep ones. The cutting edge is shown more in detail in Fig. 170, which also shows the triangular knee bracing.

The surveys for the caissons, including accurate soundings, were made from a circular raft 10 feet wide (Fig. 171) moored over the site of each caisson separately, being shifted around so that the area for about 50 feet outside of each one was mapped. The piers at Inchgarvie being on sloping rock, it was decided to level up the bottom before landing the caisson, and two rectangular piers were formed of concrete in bags (Fig. 172) and these surrounded with sand-bags to make a tight closure between the cutting edge of the pier and the bottom.

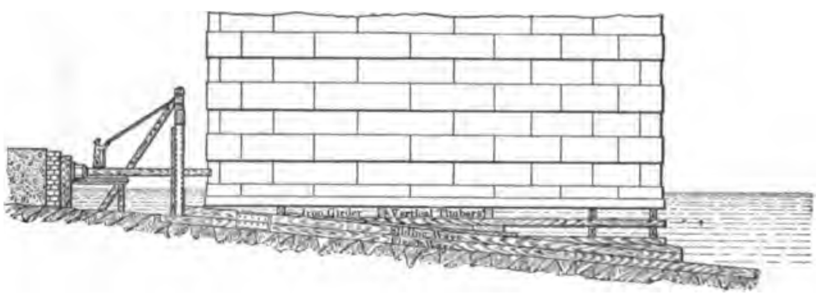


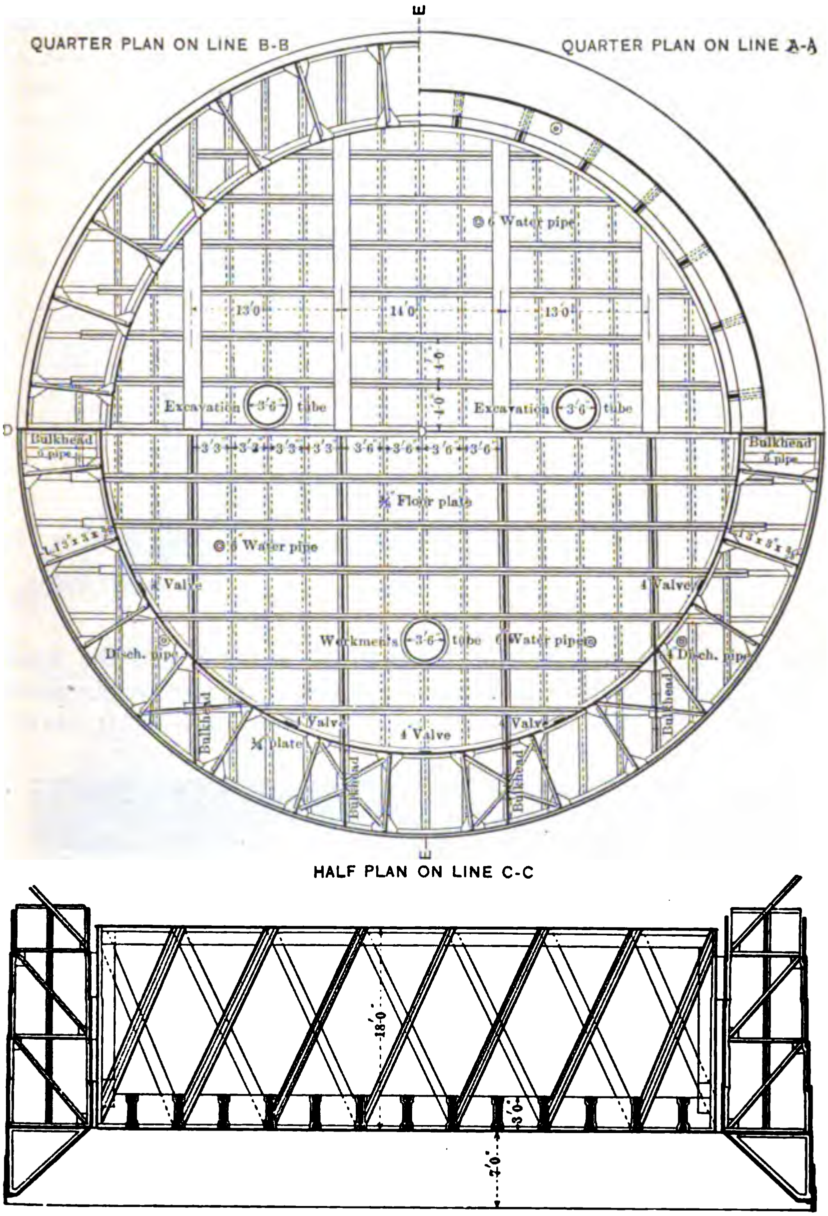
FIG. 167.—LAUNCHING WAYS, FORTH BRIDGE CAISSONS.

The placing of these bags was accomplished by divers, and they were continuous around the circumference, except in several places where openings were left for the air to escape and for the discharge of débris during the sinking. The caissons were provided with three shafts 3 feet 6 inches in diameter, two for material and one for the workmen. There were also provided three 6-inch pipes for the introducing of water under pressure and also air-pipes and discharge-pipes.

The air-locks were of the type shown in Fig. 173, of circular form, and two entrance doors to the lock from the outside and two doors giving access to the ladders in the shaft. The height of the locks was 6 feet and the diameter 7 feet, setting on and fastened to the shaft, which was made up of lengths of 8 feet and connected together with angle flanges.

The material locks are shown in Figs. 174, 174*a* and 174*b*, and were attached to the shafts as shown with sliding doors at the top and

Fig. 168.—Towing Caisson to Site.



FIGS. 169*a* AND 169*b*.—METAL CAISSON DETAILS, FORTH BRIDGE.

bottom, which were worked by hydraulic rams. There was an interlocking device, making it impossible to open both doors at the

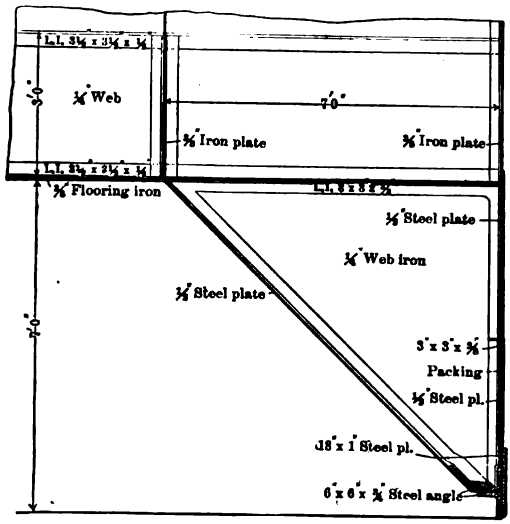


FIG. 170.—CUTTING EDGE FOR CAISSON.

same time. The engines for hoisting the material and the worm gearing are sufficiently shown, so as not to require explanation. The inlet and outlet valves of the chamber were large and it was not

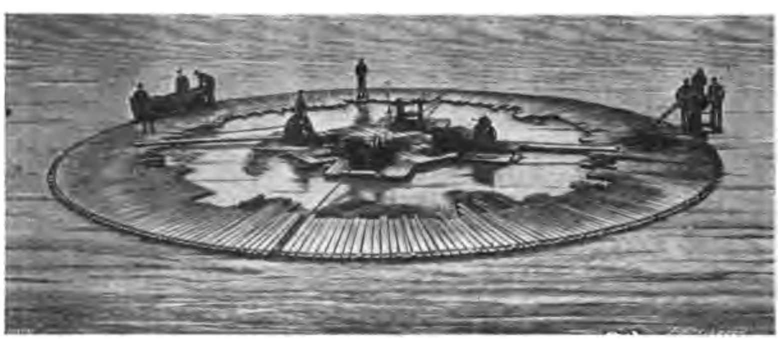


FIG. 171.—SURVEYING RAFT, FORTH BRIDGE PIERS.

necessary to make the change of pressure slowly, as no men were allowed to enter or leave the locks by this way.

The valves on the lock for the workmen were, however, of small size, so that the change of pressure on entering or leaving could be

made gradually and no danger to the men result. On one occasion the rubber gasket on one of the outer doors blew out, with one man in the lock, and the sudden change of pressure in less than two minutes caused bleeding from the ears, nose and mouth, and severe pains in the limbs, but as he was a strong, healthy man, he fully recovered. The men working under pressure were required to be

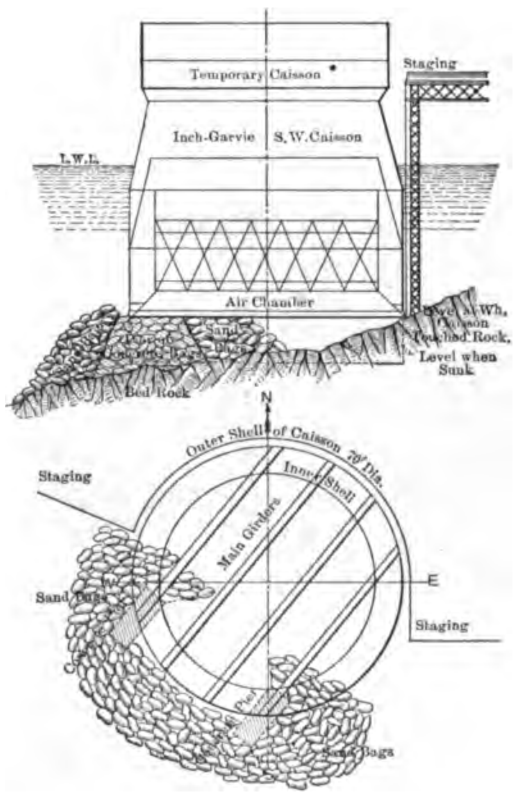


FIG. 172.—INCHGARVIE PIERS ON SLOPING ROCK.

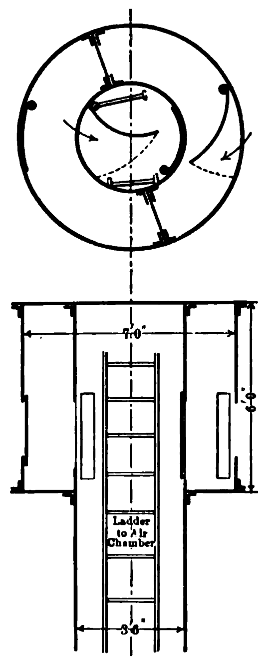


FIG. 173.—AIR-LOCK, FORTH BRIDGE CAISSONS.

sober, in good health, free from lung trouble or gastric weakness. The only two men who died were found to have been already consumptive when they began working in the caissons.

The same bad effects were observed, as already referred to in Chapter XI, where a discussion of caisson disease is given. The caissons were fitted with electric lights and everything possible done for the convenience of the men and for expediting the work.

The Queensferry jetty when completed had four recesses left for receiving the caissons, and when they were in place they were retained by heavy dolphins, and strong chains provided to attach to lugs on the steel shells. There were also provided additional ropes and cables for fastenings in case of emergency.

Previous to launching, all joints in the roof and sides of the caisson were thoroughly calked, and concrete filled in to a depth of about 5 feet over the roof, making a total weight for launching of about 400 tons. The launching ways had been laid on concrete blocks, at an inclination of 1.1 to 12 as already stated, and the caisson had been

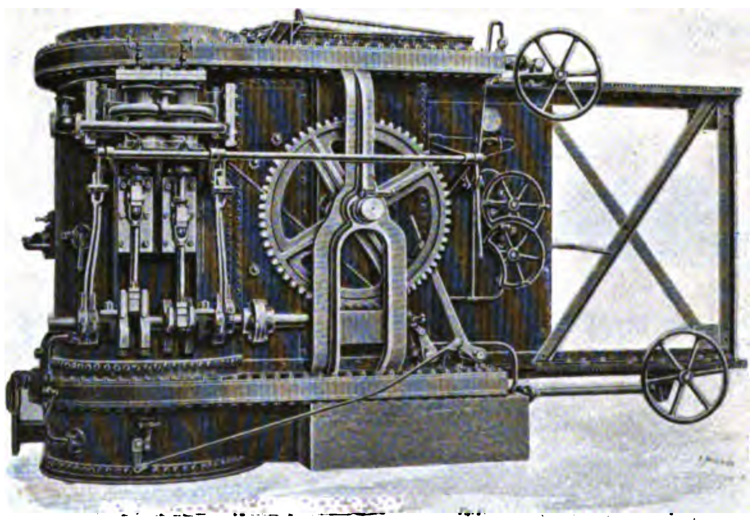


FIG. 174.—MATERIAL LOCK, FORTH BRIDGE.

lowered down when the weight reached about 300 tons, to within about a foot of the cradle (Fig. 167), as much of the cutting edge and roof being supported directly from the cradle as was possible.

When all was ready for launching the points of contact between the cradle and the caisson were as carefully adjusted as possible, the cradle weighted with iron to hold it on to the ways when the caisson had floated, then finally at or near the time of high water the caissons were set in motion by a 12-inch hydraulic jack and launched, drawing from 9 feet 6 inches to 10 feet 6 inches of water. They were then towed to the jetty to receive the additional concrete, brickwork, and all the machinery for sinking, which made the weight ready for landing about 2900 tons. This was made up of the

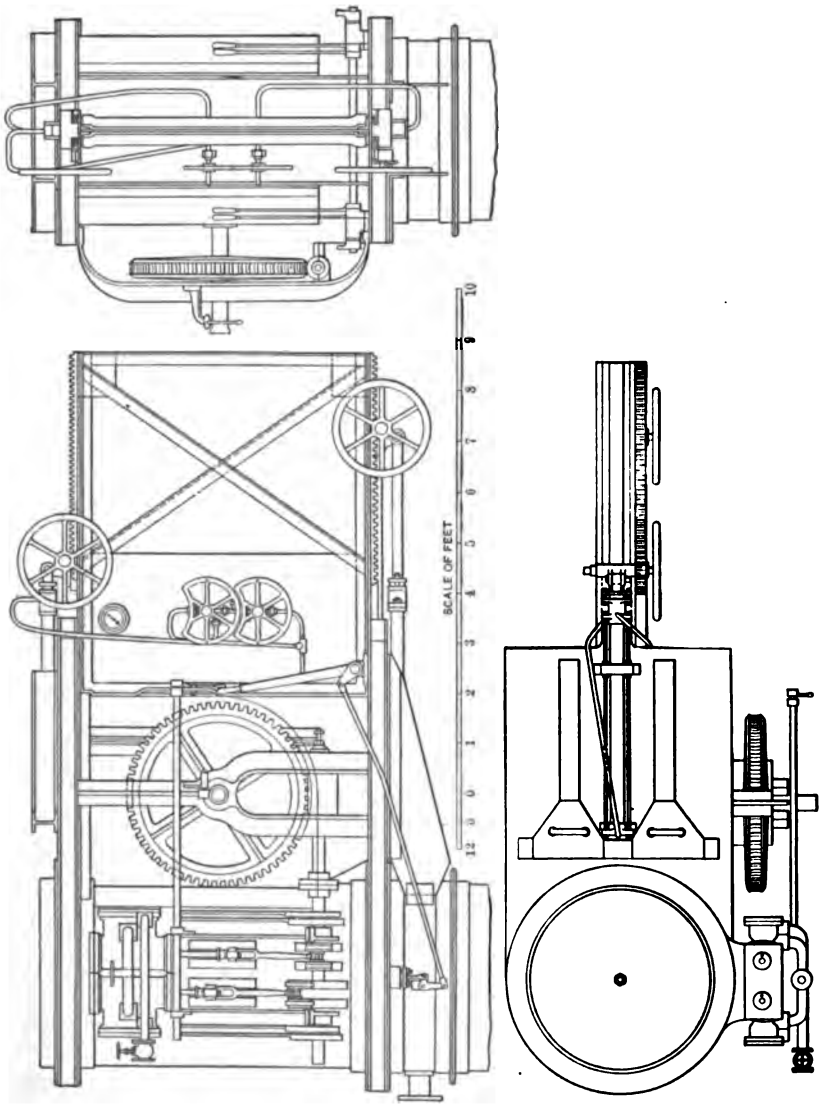


FIG. 174a.—MATERIAL LOCK, FORTH BRIDGE CAISSONS.

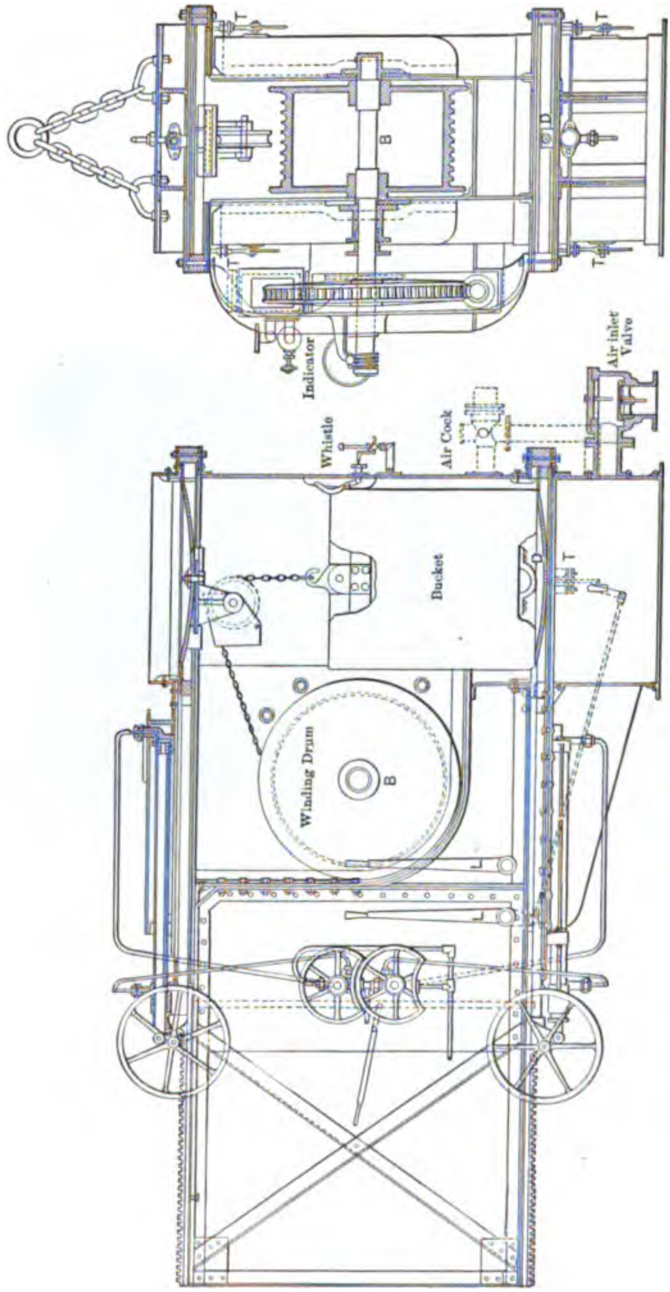


FIG. 174b.—MATERIAL LOCK (SECTIONS), FORTH BRIDGE.

launching weight of 460 tons; two tiers of temporary caisson or coffer-dam, 65 tons; air locks, shafts and machinery, 35 tons; timber floors and staging, 50 tons; concrete, 1420 tons; and brickwork, 805 tons—making a draft of 31 feet.

The air compressors used in sinking were of the ordinary English type, with two horizontal coupled engines, acting directly upon a pair of double-acting air cylinders. These consisted of two pairs of $16\frac{1}{2}$-inch diameter by 24-inch stroke cylinders; three pairs of 12-inch diameter by 24-inch stroke, all driven by 60 pounds steam pressure. The air cylinders were water-jacketed and the compressors at various speeds gave the air pressures required for different depths and conditions, the maximum used being 37 pounds per square inch on one of the Inchgarvie caissons. The compressors for the rock drills were separate and delivered air at 70 pounds pressure, but giving inside the working chamber only the difference between this and the pressure on the caissons.

The sinking of the Queensferry caissons through the soft material was accomplished without any particular difficulty, except for the northwest caisson, which underwent a bad accident as it was landed for sinking, and which will be described later.

The air shaft and the working chamber underneath was first cleared of mud, as soon as the caisson had been landed, and great care had to be exercised to preserve the bouyancy, as there was nothing else to prevent the caisson from sinking in the mud and smothering the men. The greatest trouble from this source was at low tide, as the displacement was less at this time, and as the cutting edge rested upon such soft material the men were withdrawn.

The soft material was removed through the ejectors, by first forming a basin or sump, into which water was turned from the supply pipes, to make the material more fluid. The workmen introduced one of the ejector pipes, letting in the proper amount of air so that the mud was carried along through the pipe and discharged outside the caisson. This was due to the velocity of the air being so much greater than the velocity of water would have been, although the actual pressure of the air was only equal to the head of water outside. It required much care and experience on the part of the men to keep the supply of air just right to keep the material moving, and even then the discharge was not continuous, but pulsating, sometimes a light stream and then masses of material. (Fig. 175.) This process worked satisfactorily until the hard material was reached and some other method had to be resorted to for the removal of the clay and boulder clay. Powder and dynamite were tried

without satisfactory results, and it became necessary to try some sort of an excavator.

The hydraulic spade (Fig. 176) was employed as devised by Mr. Arrol. This was operated by the water pressure from the pipes, of 1000 pounds per square inch, less the pressure of air in use at the time. With the spade set on the ground by two men and the head against the roof of the caisson, air was turned on and a slice was cut off up to 4 inches thickness and to a depth of 16 to 18 inches. These slices very often had to be broken to put the material into the skips for removal and any large boulders loosened had to be blasted for

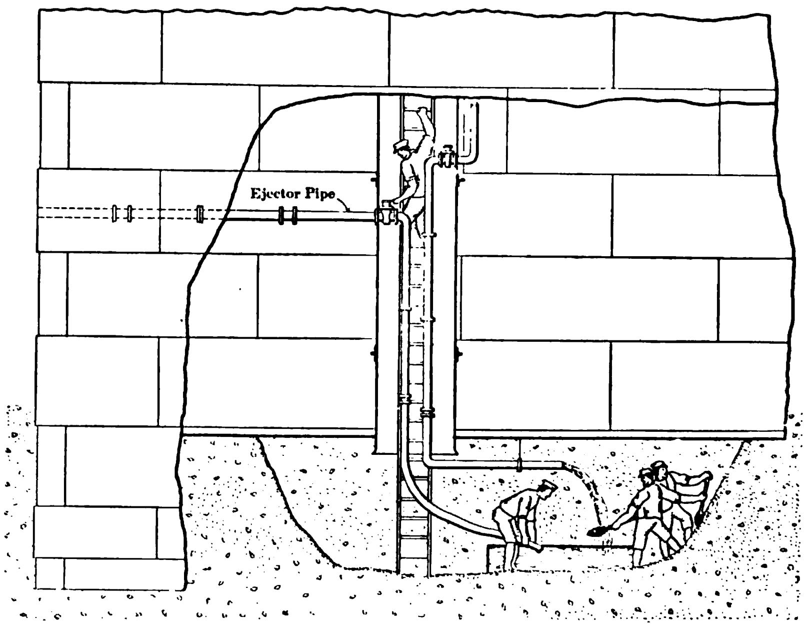


FIG. 175.—EXCAVATING AND DISCHARGING MATERIAL FROM CAISSON.

removal or left in the caisson to be concreted in when the sinking was completed. Where the spade was operated at an angle in digging out under the cutting edge, the rivet heads in the roof served to keep the head of the spade from kicking out. The material was gradually removed from under the cutting edge until it would not sustain the weight of the caisson and it gradually settled into the clay, when more weight of concrete would be added above.

After the caisson had gotten well into the clay, the water pressure from above sealed the clay against the shell and it was only necessary to carry from 15 to 18 pounds air pressure, although the depth of

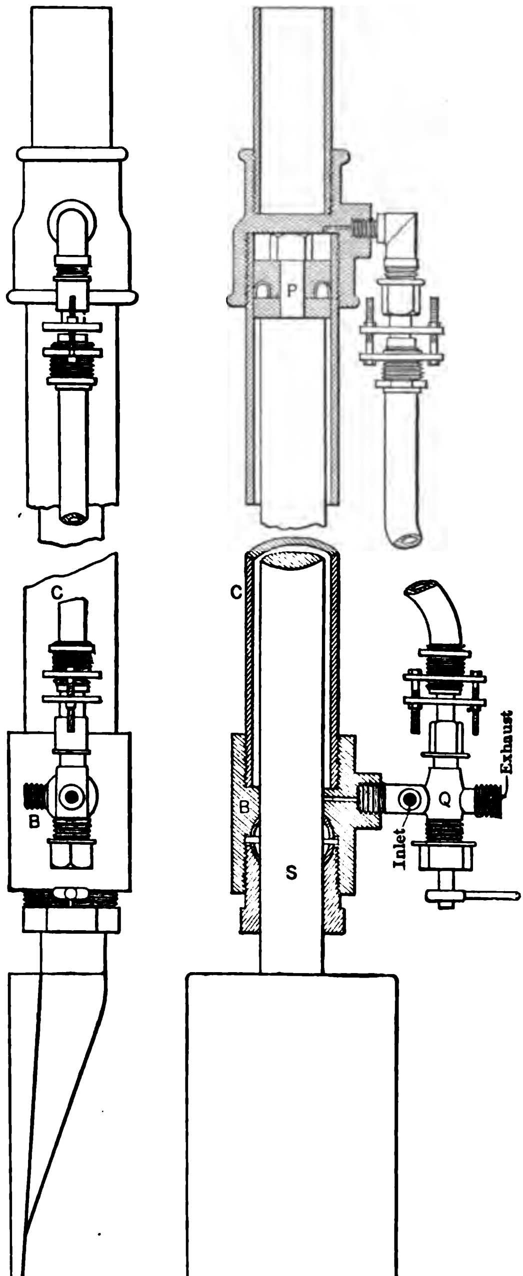


FIG. 176.—HYDRAULIC SPADE.

89 feet called for a pressure of 39 pounds to resist the head of water outside; this reduction of the working pressure made it much easier working for the men, and saved very materially the machinery used in the sinking and excavation.

The position of the caissons was watched very closely day by day and checked up carefully every few days. If they were found to be out of place the cutting edge would be undercut on the side toward which the caisson had traveled, thus tilting the entire caisson in that direction and then sunk for some distance in that position. Then the high side would be undercut to bring the caisson back to level and position. The batter of the sides of the shell materially assisted in this, and it is believed that a still greater batter would have been of greater assistance.

The greatest record for excavation for a gang of twenty-seven men, two shifts, working twenty-four hours with the hydraulic spades, was a bucket full every five minutes or over 5 cubic yards for each man in twelve hours. This would mean about 270 yards per day, or with 145 yards to excavate per vertical foot, a maximum sinking of about 1.8 feet per day, which is of course very much above the average shown in the table.

When the caissons had reached their final positions in the boulder clay, the working chamber was entirely cleared and made ready for concreting. Doors opening downwards were attached to the lower ends of the material shafts, the air locks removed and 18-inch tubes with air valves, carried up to the concrete mixers. With the lower door closed, concrete was shoveled in until the tube was nearly full, the air pressure turned on, the bottom door opened and the concrete allowed to drop into the chamber.

The men then proceeded to convey it to the sides and tamp it firmly under the cutting edges, and as the filling progressed, to tamp it under the roof. When the chamber was full, the shafts were filled, and neat cement grout run in to fill up all crevices.

Then the shells were filled up in the open, up to the level for the beginning of the granite masonry.

The sinking and filling of the two Inchgarvie caissons was carried out on the same general lines, except that the excavation was in rock on one side to begin with and in the bags of sand and bags of concrete on the other side, thus making a somewhat ticklish piece of work, that required constant care and foresight.

The only serious accident on the six caissons was on the northwest Queensferry caisson, which, being landed in shallow water, sank into the mud at half tide. Being caught by a very high tide and

then a very low tide, it sank still further into the mud, took on some list and on the high tide again coming in, the coffer-dam portion filled, causing more list, so that the caisson moved sideways about 20 feet, and deeper into the mud. It then became necessary at great expense and great loss of time to build a wooden coffer-dam on top by means of diving, before it could be pumped out and refloated. Although this caisson was launched and towed to its berth on Dec. 3, 1884, it was Nov. 25, 1886, before the sinking was begun.

TABLE XXII.—SINKING RECORD, QUEENSFERRY PIERS

	Southwest.	Southeast.	Northeast.	Northwest.
Launched and located.	May 26, 1884	Aug. 24, 1884	Nov. 24, 1884	Dec. 3, 1884
Submerged..........				Jan. 1, 1885
Floated second time..				Oct. 19, 1885
Sinking started......	Sept. 1, 1884	Nov. 24, 1884	Jan. 28, 1885	Nov. 25, 1885
Depth below H. W....	33 feet	33 feet	37 feet	52 feet
Sinking completed....	Dec. 6, 1884	Feb. 4, 1885	April 10, 1885	Feb. 4, 1886
Final depth..........	71 feet	73 feet	89 feet	85 feet
Average sinking per day...............	4.70 inches	6.67 inches	8.67 inches	5.58 inches
Material encountered.	Soft 15 ft.	19.5 feet	38 feet	10 feet
	Hard 23 ft.	20.5 feet	14 feet	23 feet
Total yds. excavated..	6372	6651	6827	6271
Concreting begun.....	Dec. 8, 1884	Feb. 7, 1885	April 14, 1885	Feb. 5, 1886
Air chamber concrete in.................	Dec. 17, 1884	Feb. 18, 1885	April 25, 1885	Feb. 10, 1886
Concreted to granite level..............	Feb., 1885	April, 1885	June, 1885	March, 1886

The foregoing account is condensed from *Engineering*, and gives all the salient features of the work. The description of the circular granite piers is taken verbatim from the same source.

The foundations below low-water level, of ten out of the twelve circular piers, vary both in size and considerably in depth, but above that level they are exactly alike. The two exceptions are the two north piers on Fife already described. Their foundations start at 7 feet below high water, and they are only 45 feet in diameter under the necking course. In all other respects they do not differ from the remaining ten piers. In all these the granite masonry starts at low-water level or, 18 feet below high-water level, with a diameter of 55 feet; rises with a regular straight batter of 1 inch in 10½ inches to a height of 12 feet 8 inches above high water, where the diameter is 49 feet, and terminates in a necking and a coping course with a somewhat rounded top at exactly 18 feet above high water. The courses of granite, of which there are nineteen,

are rock-faced, while necking and coping are of dressed granite, vary in thickness from 21 inches in the lower to 16 inches in the upper courses, while above these the necking is 19 inches and the coping about 3 feet 6 inches thickness. The latter two courses are of Cornish granite, the others of Aberdeen granite, both light gray

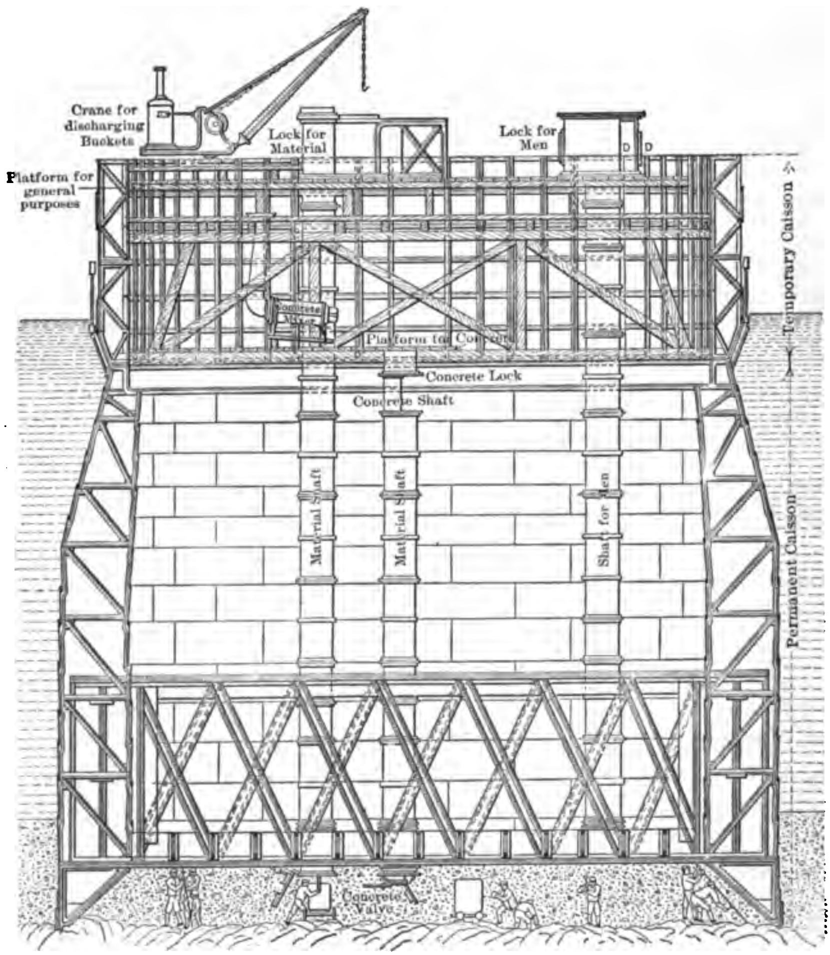


FIG. 177.—SECTION OF CAISSON WITH AIR-LOCK, FORTH BRIDGE.

in color. The blocks of rock-faced granite have the edges dressed to the batter of the pier, and to horizontal and vertical joints. The courses are alternately headers and stretchers, with a bond of not less than 9 inches. The joints are not more than $\frac{1}{4}$ inch wide, and are pointed from the outside with pure cement.

The hearting is principally of flat-bedded Arbroath rubble, but a large number of whinstone blocks roughly squared were also built in. In building the pier the rubble masonry closely followed the setting of the granite, and both vertical and horizontal bond was strictly observed. Between the concrete in the foundation and the rubble masonry in the piers bond was also established by large blocks of whinstone squared to obtain proper bedding. (Fig. 109.) At this point a wrought-iron belt 56 feet 6 inches in diameter, 18 inches in depth and $\frac{3}{4}$ inch thick, made in sections and riveted together, was placed and built in. A second belt of similar strength, but only 43 feet in diameter, was built in about 2 feet below high water, and a third belt of double strength, or $1\frac{1}{2}$ inch thick and 39 feet in diameter, was built in just behind the coping course. All these are shown in Fig. 109.

When the level 7 feet below high water was reached a temporary timber stage was erected and carried to the level of the under side of the fixed bedplate about 17 feet 6 inches above high water. Upon this a templet of the bedplates made of light angles and $\frac{1}{4}$-inch plate in four sections bolted together, which had all the holes for the holding-down bolts in it, was laid, correctly centered and screwed down. The bolts with anchor plates supported by nuts were then carefully set up and built into the rubble masonry, a space of a few inches being left round each bolt to admit of subsequent adjustment. This is very distinct in the Inchgarvie northeast pier. The building of the pier was now continued, the position of templet and holding-down bolts being frequently checked to insure correctness.

From the plan it will be seen that the coping course consists of alternate headers and stretchers.

The top or crown of the pier is slightly spherical, and is built up of blocks of dressed Aberdeen granite from 17 to 18 inches in thickness. The blocks fitting into and adjoining the coping course were all cut to wooden templets sent, the remainder being arranged in straight courses. The blocks project from 6 to 12 inches under the bedplate, a recess being cut out to receive the latter. The remaining space under the bedplate is made up with Arbroath rubble masonry, and in one case with a tolerably thick layer of Staffordshire blue bricks built in cement.

The mortar used in building the piers was throughout of 1 part cement and 2 parts of sand, and was mixed in a pugmill close by. At first hand cranes only were used in the building, but steam cranes were substituted as being more handy and expeditious. Both

granite blocks and rubble were handled and set by means of pointed chain-clips, holes being picked for the purpose.

The holding-down bolts in each pier were forty-eight in number, in four rows of twelve each. They are set at 2 feet 9 inches and 7 feet respectively to each side of the center line, and are longitudinally about 3 feet apart. Owing to the inward set of the bottom members in cantilevers, rather more than half of the bolts are set in a line parallel to the center line of the bridge, the remainder following the deviation of the bottom member. In order to bring the largest possible mass of masonry into play, the four center bolts of each outside row are bent outward, as shown in Fig. 109, and to prevent any tearing action upon the masonry which would be produced by the bolts being drawn tight at top, cast-iron shoes are inserted at the point of kinking, and these are held together by a pair of angle-bars to each pair of bolts. The holding-down bolts are of a special steel. They are 2½ inches in diameter, with an enlargement to 3 inches at both ends, where a screw thread is cut upon them. They are about 25 feet long. The anchor plates are 2 feet square with a long boss, stiffened by four diagonal ribs, and are held by an ordinary nut. The bolts received several coats of tar before being built in.

When the masonry had been carried to within about 8 feet of the top the templet had to be removed to allow the rubble to be placed in position, but it was replaced from time to time, and the position of the bolts frequently checked. As the heads of these bolts fitted in the lower bedplate without any play whatever, it was necessary that their position should be absolutely correct.

When the masonry had got up close to the under side of the bedplate the bolts were again set with the greatest care, and the spaces left round them were filled up with cement grout to within about 4 feet of the top.

Immediately underlying the bedplate, and with a view of making a perfectly level bed for the same, cast-iron blocks, 12 inches square and 4 inches thick, with a hole which only just admitted the head of the bolt, were placed, carefully leveled by instrument and set in cement, the spaces all round and between these being leveled to the thickness of 1 inch with cement. In addition to the round hole in each block, a slot was cut and a taper wedge driven hard into this and against the screw thread. This was done to prevent torsion in the bolts when the large upper nuts required to be drawn tight. On the bed thus prepared the lower bedplate was laid in the manner hereafter described. The heads of the holding-down bolts are shown in connection with the bedplates.

CHAPTER XIII

DIVERS AND DIVING

THERE will be only occasional need for a diver's services on the usual class of foundation work, but it is very essential that such work be fully understood by the engineer, and more especially where it is possible to obtain only the diver's outfit and the diver must be created out of some employee on the work. The first essential in such a case is to know the physical characteristics that such a man must have to make a success in diving. In the first place he must be a cool-headed, quick-thinking man, with a first-class physique, not full blooded or with a florid complexion, nor with a short neck. The other disabilities which should deter men from attempting diving are constant headaches; palpitation of the heart; slight deafness or running from the ears; spitting or coughing blood; poor circulation of the blood as indicated by cold hands or feet, extreme paleness, or bluish lips; bloodshot eyes or high colored cheeks; intemperance; venereal disease; rheumatism and sunstroke.

The diver should not be over forty-five years of age; he should not eat any considerable amount before descending, and he should be supplied with the same amount of air he is used to breathing on the surface. In case the man selected shows nervousness, he should go a little way and return to the surface, and if he cannot eventually overcome his fear, he should give up the attempt; but without nervousness the descent should be made as quickly as possible. No fear need be felt of accident unless something is caught; for in case of the hose breaking or getting cut there is air enough in the helmet to last about five minutes.

The ascent should be made at the rate of about 1 foot per second up to within about 50 feet of the surface, where a short stop should be made, and the ascent from there up, or in water 50 feet or less, very slowly. Should he suffer from coming up too rapidly, the application to pressure again will give relief. The general effects of compressed air are not harmful, as many men are rendered more

healthy and their digestion better by the increased amount of oxygen they absorb in the compressed air.

The pressure due to a depth of 30 feet is 13 pounds per square inch, and that for other depths to the nearest pound may be taken from the following table:

TABLE XXIII.—AIR PRESSURE, LBS. PER SQ. IN., FOR VARIOUS DEPTHS

Depth.	Pressure.	Depth.	Pressure.	Depth.	Pressure.
30	13	100	43	170	74
40	17	110	48	180	78
50	21	120	52	190	82
60	26	130	56	200	87
70	30	140	61	210	91
80	34	150	65	220	95
90	39	160	69	230	100

The rough rule used by divers and workmen is one-half pound of air per vertical foot.

The usual maximum limit for deep work is 150 feet, and at this depth the diver will ordinarily stay down a very short time, likely not over twenty minutes. In one case where a tug had sunk in New York Harbor in 150 feet of water, one diver was sent down to attach a chain to the shaft for the purpose of dragging the boat into shallow water. As he did not answer the signals, he was hauled up and revived with difficulty. Another diver, a small, wiry, hardy man, went down several times, staying as long as twenty minutes a trip, without bad effects, and succeeded in attaching the chain. Of course below a depth of 30 to 40 feet, there is not light enough to do much work without artificial light, and in New York Harbor it becomes dark below 20 feet.

The earliest deep diving was that done by the pearl divers in the Indian Ocean, without any suits or assistance, but they could remain down for only two or three minutes at the most, and any stories to the contrary may be taken as pure fiction. The use of a diving dress was first described by Dr. Halley in 1721, and the modern type of dress was gotten up by Siebe about 1829, but it was open at the bottom, so that if the diver fell or leaned over too far the air spilled out and he would be in serious danger unless hauled up at once.

The most ordinary depths for diving are from 30 to 60 feet, but there is a case of very deep diving on record, by Hooper, off the coast of South America, who made seven descents to a depth of 201 feet, or at a pressure of 87 pounds per square inch, remaining down one trip for forty-two minutes. The record on Puget Sound in salv-

ing a boat is a depth of 185 feet, where the diver made many descents to a maximum time on any one trip of about one hour, but he afterward died as the result of the severe experience. Lights at such depths burn out very quickly, and in caissons candles and electric lights will burn out, where the depth is 100 feet, in three-fifths of the time as at the surface. If a candle is blown out at 80 feet in a caisson it will immediately relight, and at 108.5 feet it can be blown out as many as thirteen times in a half minute and relight each time.

The ordinary diver picked up for only one short piece of work will charge from $50 to $75 per day, and his reports should be checked up by soundings or in some way, as they are often not reliable in the data they give; so that the best way will be to get a diver of reputation or one well recommended. Where one is needed constantly around a large piece of work, he can usually be employed at about $5 per day about the upper work and at from $10 to $20 for the days he is diving. His helpers should also be employed on the work constantly, so that he will have experienced men to handle the pump and lines. Such an arrangement will tend toward the diver giving more reliable information, as he will not be seeking to lengthen his job. It must be borne in mind that a diver can exert but little if any downward force in working, and in exerting a horizontal force he must brace himself with his foot, or lacking a foothold he must brace himself with a line around a timber or pile.

The recent invention of an oxyhydrogen blowpipe by a German for use under water by a diver is a great help for working on or removing metal work. The great trouble in a device of this kind is to prevent the water from putting out the flame. This is prevented (Fig. 178) in the ordinary Greisheim burner, by adding a bell-shaped hollow head and by air under pressure. This was recently used in the harbor at Keil, Germany, and the diver made cuts 4 inches long, without difficulty, and burned out a surface $44\frac{3}{4}$ square inches in area. Working under water, another diver, in a depth of 16 feet, made a cut 10 inches long in a minute and a half, and in half a minute cut through a plate about $\frac{7}{8}$ of an inch thick.

The experienced diver can do ordinary woodwork with hand tools under water, and can accomplish great results with powder. Three sticks of dynamite placed around a pile at the ground surface, and shot off by a battery after the diver returns to the surface will cut it off clean, but no less than three sticks will be effective on an ordinary sized pile, and a large one will require four. The diver can easily shoot off from forty to seventy piles in one day. Four or five sticks of dynamite placed on a large chunk of concrete will

completely shatter it, requiring only fifteen or twenty minutes of a diver's time.

The effect of such shots under water will not jar surrounding structures to damage them, nor in fact will shots of thirty or forty sticks, unless placed so close to a pier or other work as to actually act on it direct. The best way is to start on any work with a few sticks and gradually increase the charge, but stop before the jar becomes hard enough to be questionable. Any adjacent structure

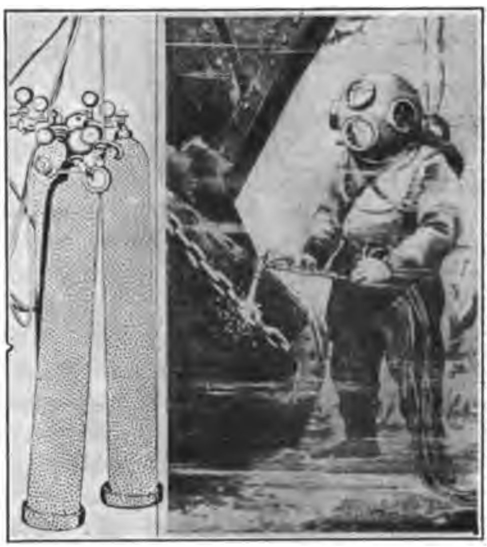


FIG. 178.—DIVER USING OXYHYDROGEN BLOWPIPE.

which will be seriously damaged by the jar from a few sticks of dynamite should be taken down or should never have been built.

The experience actually encountered by a diver in salvage work is the very best discussion of what they can do, and the following article of diving work off the English coast gives good ideas of the methods of diving and its dangers:

Mr. Anton Graf, the narrator of this story, claims to be the oldest active diver in the world. During the course of an experience extending over a period of nearly sixty years he has naturally met with some thrilling adeventures, one of which is here described.

"In the middle of August, 1879, there was a terrible storm in the English Channel, the sea being so rough, and the tide rising to such an extent that in some of the streets of Dover the water was two or three feet deep. For some days previous to the storm I had been employed by the Coast Wreckage Company

on salvage work in connection with a steamer which had sunk about twelve miles off Dover. The severe weather, however, caused a temporary stoppage of all diving operations, and I received orders from my employer to go out with the steamship *Harold* from Dover to render assistance to any ships found in distress.

"Before we had been in the Channel very long we saw a steamer collide with a bark below Folkestone, the sailing vessel being so badly damaged, having been struck amidships, that she quickly sank. The bark was loaded with salt and empty petroleum barrels, and as soon as she disappeared beneath the waves many of the barrels came to the surface, dotting the sea all round us. We did our utmost to save the crew, but were only successful in rescuing the steward's boy, whom we noticed floating on a half-barrel near the scene of the disaster.

"The steamer was the *Idlewild*, owned by the Navigation and Transportation Company, and carried a cargo of iron ore. She had also been considerably damaged, her bows being much bent. Her captain decided to try to reach Folkestone, but his attempt was futile, for the water rushed into the vessel in great streams. When we saw the precarious condition she was in, we asked the captain to close the bulkheads, which he did, and we then towed the steamer into Dover Harbor.

"The sunken bark, an American ship named the *Delson*, had foundered about fourteen miles west of Folkestone, and when the gale subsided, about two days later, I received instructions, through the Trinity Commissioners, to blast the masts out of the ship, in order to prevent other vessels coming into collision with them.

"I was informed that there was a good deal of jewelry and money in a locker in the captain's cabin, and I determined to obtain these valuables after I had carried out my other orders.

"The blasting work did not take very long, three charges of dynamite proving sufficient to get the masts out. As soon as I had finished these operations I commenced a search for the desired cabin. I quickly found it, and also the locker in which, in all probability, the valuables were located. The drawer, however, was locked, and I went back to the deck to search for a piece of iron in order to force it. I discovered what I wanted hanging from one of the davits, and soon twisted it free. In returning to the cabin, I had to be very careful to keep myself free from the stanchions, so that I should not get my lines fouled when I wanted to regain the deck.

"It took me some time to wrench open the drawer, but I eventually succeeded, and on putting my hand in I found some paper money, a watch and chain, some rings, and a lot of coins. These I put into one of the big pockets of my protection overalls, and then turned to make my way back to the door of the cabin.

"To my surprise, however, no light filtered through from the direction of the door, although, after I first entered, the opening could plainly be seen. I had evidently been too intent on my task to notice the change. The cabin was now in almost complete darkness, and I knew that in making my way back I should have to be very careful to steer clear of obstacles. I rolled my hose and lines round my arm, and commenced to grope my way to the three steps which led up to the door. These I reached with little difficulty, but when I tried to push the door open I found to my consternation that I could not even move it the fraction of an inch.

"I must explain here that in the part of the Channel where I was working it is only possible to carry on diving operations for about four or five hours during

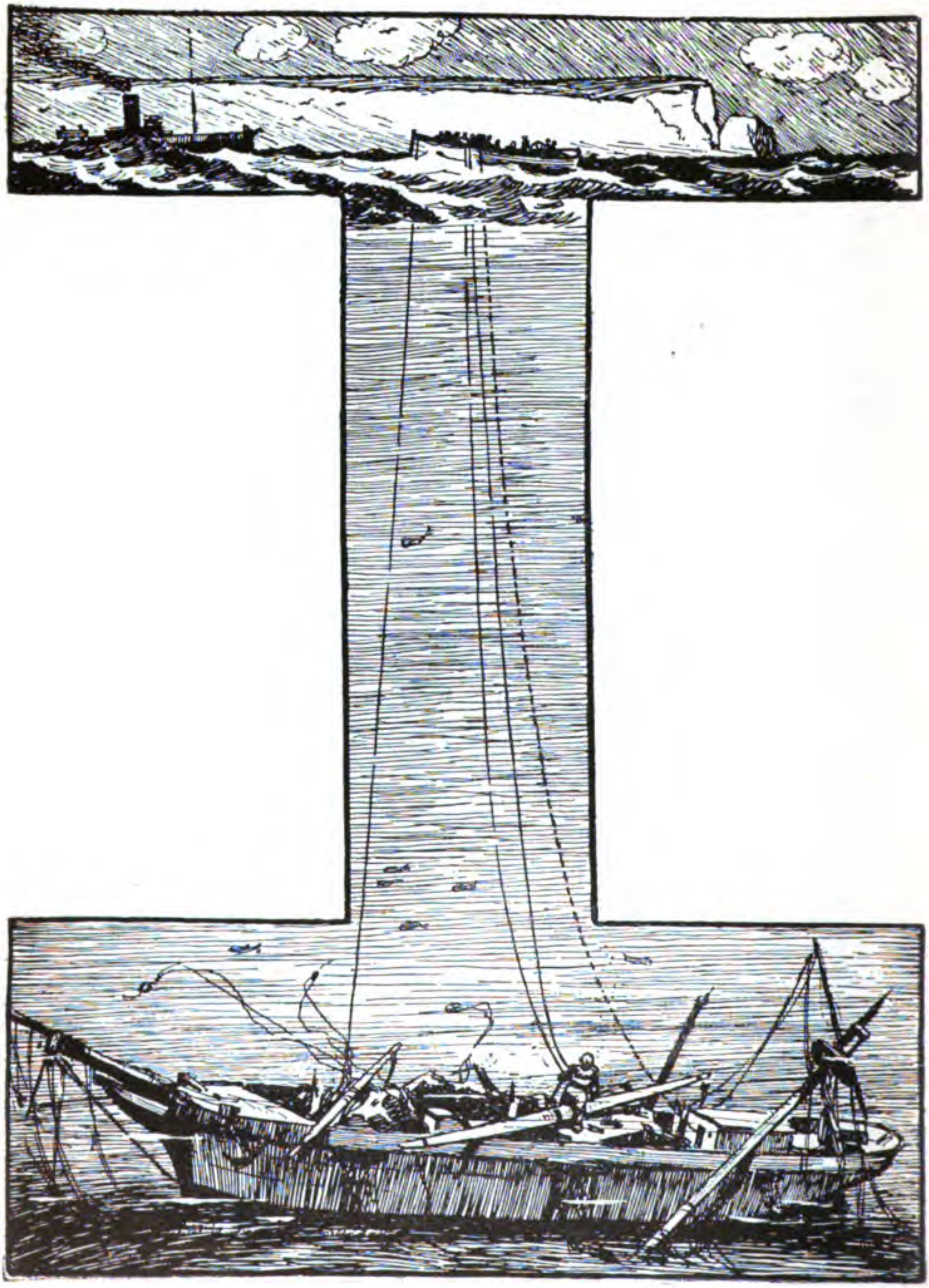


FIG. 179.—DIVER AT WORK ON WRECK.

slack tide. At other times the currents are so strong as to make diving a very hazardous undertaking and practically out of the question. When I entered the cabin the tide was flowing down Channel, and in consequence the current kept the cabin door open. I had been so occupied with the task of collecting the captain's valuables that I had failed to notice how quickly the time seemed to go. A strong current was now running in the opposite direction, and pressing with tremendous force against the door, keeping it as securely closed as though heavy bolts were assisting the relentless currents in their fight against me.

"The horror of my predicament now dawned upon me, and I could not help thinking that I should be extremely fortunate if I ever again saw the blessed light of day. It was obvious, however, that inaction would not improve my prospects, so I started to feel for the spot where my hose and lifeline passed between the door and the framework. I found them at last—over the top of the door—and, luckily, the tough lifeline was nearest to the hinges, thus keeping the door slightly more ajar than would otherwise have been the case, and so protecting the hose. If the positions had been reversed, there is no doubt that the hose would have been jammed so tightly as to prevent any passage of air through the tube, thereby causing my speedy suffocation. Even as it was, the line only partially saved the hose, and within a minute or so I began to experience great difficulty in breathing—a hint of the agony to come if I did not speedily extricate myself.

"For a moment I was nonplussed as to what I should do next. I knew only too well that if I found it impossible to escape from my submarine prison, I should not remain conscious very long. I was, of course, aware that those at the top would quickly come to the conclusion that something was amiss if I did not ask to be hauled to the surface. But I was quite unable to imagine what means they could discover to effect my rescue and as my lines were jammed I could not signal to them in any way to explain my position. There was no other diver on the tender—the *John Bull*—and it was impossible for them to send to Dover for assistance, even if there had been time to do so.

"Anyhow, there was no time to be lost, as the air in my helmet was already getting very close, and it did not seem as though much more was filtering through the narrow piece of tubing wedged in the tight-shut door. Turning round, I descended the steps to try to find an instrument which I could use as a lever in my efforts to force open the door. If I could only move it an inch or so it would probably enable me to give a signal, and, by releasing the hose somewhat, allow more air to reach me.

"For two or three minutes I groped about for the piece of iron with which I had forced the locker, but nowhere could I find it. Fortunately, however, my knuckles struck the handrail leading up to the door, and seizing this with both hands, and placing one of my feet against the wall, I endeavored to wrench it free. The strain was so great, especially as I could hardly breathe, that I was afraid I might burst a blood vessel, but at last the lower part gave way, and it did not then take me more than a moment to release it altogether. The exertion, however, had told on me severely, and I could do nothing further for a minute or two, although I fully realized that every second's delay only added to my danger.

"Eventually, however, I managed to insert the rail between the door and the framework, and, using it as a lever, gradually succeeded in forcing the door slightly open and letting down the hose and line. Now that the pressure was somewhat relieved the air came through more freely, but very jerkily,

although this was not to be wondered at, considering that the tide was running at the rate of twenty-two miles an hour. My nerves were in an awful state by this time, and more than once I came very near to suffocation.

"I now tried to signal to the tender, and to my delight found I was able to do so. I asked for a chain to be attached to the lifeline and sent down, but the answer I got was 'Come up.' I replied with the signal for 'Trouble,' and almost immediately the desired chain was let down. But here a new difficulty met me. The tide was so strong that my lifeline, instead of being fairly straight, made a wide curve, and the chain did not come within several feet of me. I therefore gave the signal to haul in the line, as I felt that if those above pulled it tight, and then sent the chain down again, I might possibly reach it. This call was also responded to, but with the result that I was nearly jerked off my feet as soon as the slack line was pulled in.

"A few seconds later I again asked for the chain to be lowered, and this time I was just able to grasp it and pull it through the opening. I then secured it to the door as firmly as possible, and once more gave the signal 'Haul in.' I learned afterwards that twelve men pulled on the chain, and I assisted them with my shoulder as far as possible. Something had to give, the door or the chain, but I knew, if it was the latter, that nothing on earth could save me from a terrible fate. With feverish anxiety I awaited the result, and it is easier to imagine than to describe my joy when I noticed that, inch by inch, the door was yielding. At last I had sufficient space to get my helmet through the aperture, and my body was not long in following. I had been confined in the cabin for an hour and a quarter.

"But my troubles were by no means over even now, although I had very little energy left. Through the changing of the tide, all the loose rigging and wreckage had swung round, and now completely blocked my way. I went back into the cabin for the handrail which had proved of such inestimable service to me, and with its assistance I cleared a path as best I could in the direction of the davits, where my moorings were made fast. Evidently the surface of the water above was not very smooth, for each time the tender moved with the waves the davits seemed to jump, causing me to think every moment that something vital would give way.

"I did not waste a second more than was necessary before starting to work my way up hand over hand, and I soon got entirely clear of the drifting rigging. As I went higher I felt the full pressure of the tide, which was running so fast that I could not keep my feet down, and soon I was confronted with a new danger, as, if my head went lower than my body, I once more ran the risk of suffocation. This unwelcome fate seemed more than probable, as I was now getting more air than I required, and my head throbbed like a steam engine.

"I clung as tightly as I could to the line, but the strain became so great and the pain so intense that I thought my fingers would part from my hands. By this time I was extremely feeble, and at last my muscles became so overtaxed that I could not hold on a moment l nger. I can dimly remember letting go, drifting away, and slowly rising up towards the surface, which I eventually reached head downwards.

"The next thing I recall is opening my eyes about an hour later on the deck of the *John Bull* in Dover Harbor. I had a splitting headache, and for nearly a week afterwards suffered from a confused and dizzy feeling. Apart from this, no ill-effects followed my terrifying experience, and two or three weeks later I again resumed my hazardous work beneath the waves."

The best diving outfit will cost about $900, exclusive of any scow, rowboats or launches, and the following list is for a Morse outfit of this type:

TABLE XXIV. COMPLETE DIVING OUTFIT

COMPLETE IN ALL RESPECTS FOR ONE OR TWO DIVERS AS SUPPLIED FOR GENERAL USE OF CONTRACTORS AND DIVERS

1 Air-pump, No. 1. Two Cylinders, Double Action with Two Patent Indicating Gages to denote the air pressure and depth of each Diver; with Water Cistern, Two Fly-wheels in Ash Chest, with Iron Rings for lashing	$500.00
These Pumps have Removable Tills fitted into the Pump Cases, in which are furnished and packed the following small parts:	
1 Oil Can. 1 Nut, for securing pump handles (spare).	
1 Union Joint, double male. 1 Union Joint, double female.	
1 Socket Wrench. 3 Double-ended Spanners.	
1 Overflow Nozzle. 1 10-inch Monkey Wrench.	
1 Screw Driver. Spare Valves, inlet and outlet.	
12 Washers, for Air Hose (spare).	
1 Improved Diving Helmet, Three Lights, Sectional Screw, to receive air in the Head-piece, or One to receive air in the Breast-plate; either style, including Safety Valve, Adjustable Regulating Valve and Recessed Gasket Seat	100.00
2 Rubber Diving Dresses; Size No. 2	100.00
150 feet Standard White Air Hose (three pieces) with couplings, at 40c.	60.00
1 set Diving Weights, belt pattern	22.00
1 pair Diving Shoes, with lead or iron soles	15.00
2 pairs Rubber Diving Mittens, at $5.00	10.00
1 pair Rings and Clamps	5.00
1 Life or Signal Line (150 feet)	2.50
1 pair Cuff Expanders	5.00
1 Knife, Belt and Air-hose Holder	10.00
6 feet Snap Tubing, at 60c	3.60
1 pair Chafing Pants	4.00
1 Helmet Cushion	3.00
2 pairs Divers' Stockings, at $1.25	2.50
2 Woolen Shirts and Drawers, Style "W," at $1.50	6.00
2 pairs Woolen Mittens, at $1.25	2.50
1 Woolen Cap	1.25
1 Basket for Helmet, Dresses, Hose, etc.	18.00
6 Extra Bolts and Nuts for Helmet (spare), at 25c	3.00
1 set Extra Couplings (spare)	2.00
1 yard Rubber Cloth for repairs	2.50
1 can Rubber Cement for repairs (1 lb.)	.75
1 Cutting Punch	.75
Complete Outfit for One Diver	879.35
Complete Outfit for Two Divers will include duplicate of each of the above items except the pump	1258.70

Fig. 180.—Diver's Air-pump.

Cheaper outfits, with less equipment, may be purchased as low as \$300 to \$600.

The air-pump (Fig. 180) is of the double-acting two-cylinder type, with water cistern, two flywheels and encased in an ash-wood chest. Pumps with kerosene, gasoline or alcohol engines can be purchased instead of the hand operated, for about \$300 additional. Steam compressors are very often used, and will require the addition to the outfit of a boiler, receiver, the proper piping and the necessary reducing and regulating valves to supply air at the right pressure and in the right quantity. Such an outfit will cost about

FIG. 181.—DIVING SCOW AND DIVER.

\$600 to \$800 additional to that given in the list. The necessary scow and house can be built for less than \$500 (Fig. 181), two rowboats for \$40, and a launch for anywhere from \$300 upward, or with a large launch, costing from \$2000 to \$3000, the entire outfit can be placed on board.

The pumps when lying idle should be kept clean and well oiled with neatsfoot oil, which will also keep the piston leathers in proper condition. The cylinders of the regular diving pump should be slightly warmed with warm water in cold weather, before the pump is started. Where the pump is run by an engine, the handles should

FIG. 182.—DIVER IN FULL DRESS.

Drellishak of the U. S. Navy, in 1914, dived 274 feet in Long Island Sound by using compressed air from a naval ship and using two torpedo tubes holding 11 cubic feet of air at 2100 pounds pressure. One hour and twenty-nine minutes was consumed in coming to the surface. Crilley of the U. S. Navy, in 1915, dived by the same means 288 feet in Honolulu harbor for the lost submarine F4, and took one hour and forty-five minutes for the trip. Both divers used ordinary suits.

be removed. Should the pump stop while the diver is down, the reservoirs must be large enough to bring him to the surface.

The diver in full dress is shown in Fig. 182; the helmet having a top light, although the ordinary helmet (Fig. 183) has no top light, but is provided with the regulating escape valve which extends inside the helmet, with a button against which the diver can push with his head to let out the air. The helmet with the top light is preferable, as it allows the diver to look up without throwing his body backward. The helmets shown screw onto the breastplate, those fasten-

FIG. 183.—DIVER'S HELMET WITH SOLID TOP.

ing on with thumb-bolts not being in favor. Telephone outfits are provided to attach to helmets, but many divers refuse to use them, preferring to use the old system of signaling with the life line and air-pipe. These signals are as follows:

TABLE XXV.—DIVERS' SIGNALS

Life Line.	Air-hose.
1 pull, all right	1 pull, enough air
2 pulls, as diver instructed.	2 pulls, more air
3 pulls, as diver instructed.	3 pulls, as diver instructed.
4 pulls, coming up	4 pulls, haul up diver

EXPLANATION OF SIGNALS

"The attendant is the responsible person and must attend strictly to his business all the time the diver is down; occasionally he should give one pull on the life line and the diver should return the signal by one pull, signifying all right.

"If the signal should not be returned the diver must be hauled up, but if the diver wishes to work without being interrupted by signal, he gives one pull on the line independently, for 'All right, let me alone.' If the attendant feels any irregular jerks, such as might be occasioned by the diver falling into a hole, he should signal to know if he is all right, and if he does not immediately receive a reply he should haul him up at once. If from any cause the diver is unable to ascend the ladder and wishes to be hauled up he gives four sharp pulls on the life line. If while being hauled up, the diver gives one pull, it signifies 'All right, don't haul me any more.' The diver should be hauled up slowly and steadily. If the attendant wishes the diver to come up to the surface he gives four sharp pulls on the life line, on which the diver should answer 'All right,' and return to the foot of the ladder and signal to be hauled up.

"If a telephone or speaking apparatus is used the diver can converse with his attendant and give such directions as he wishes."

The waterproof diving-dress is shown in Fig. 184; it is made of white cloth and is reinforced in all parts most exposed to wear, the sizes running from No. 1 for a 5-foot 6-inch man to No. 4 for a 6-foot 2-inch man. Various modifications of diving-dress are shown in all the catalogues.

The shoes ordinarily furnished (Fig. 185) have lead soles and brass toe caps, although they can be obtained in many varieties; with iron soles and toe caps or brass soles and toe caps, and heavy rubber overshoes can be used over these. The lead weights (Fig. 186) are riveted to a broad waist belt and carried by straps over the diver's shoulders. The knife-belt (Fig. 187) is provided with a knife going into a sheath with spring lock.

The care of a diving-dress as given by the best authority on this subject is quoted in full:

"Diving-dresses should never be packed away wet or damp. They should be well dried inside and outside or they will mildew and rot.

"If the dress has been urinated in it should be turned and well washed with clean water and left until thoroughly dry. If used in salt water it should be washed at least once a week in clean fresh water and allowed to become thoroughly dry.

"The following method for drying a dress will be found very effective:

"Take two pieces of wood about 8 feet long and nail together in the form of a St. Andrew's cross. Place inside the dress and place another piece through the arms to keep them extended. The dress can then be placed in an upright position until it is dry."

FIG. 184.—DIVING DRESS WITH MITTENS.

FIG. 185.
DIVING SHOES.

FIG. 186.
LEAD WEIGHTS FOR DIVER.

The repair of a diving-dress is carried out much the same as the repair of any waterproof fabric:

"First.—The defective portions to be cemented must be thoroughly dry and free from dirt.

"Second.—Lift surface cloth by loosening with benzine, then roughen the under rubber surface with sand or emery paper.

"Third.—Coat exposed roughened rubber surface with three coats of pure gum cement, allowing each coat to dry thirty to forty-five minutes before the next application.

"Cut the patch about one inch larger on all sides than the rubber exposed surface to be patched, remove sheeting wrapping protection cloth from the patch, loosening, if tightly adhered, with benzine, then swab exposed rubber surface with benzine. Apply one thin coat of pure gum cement, allowing same to dry thirty to forty-five minutes.

FIG. 187.—KNIFE BELT FOR DIVER.

"Next lay the edge of patch on exposed rubber-cemented surface of the dress, then gradually work the patch down on to the dress, using the fingers so as to remove all wrinkles and air bubbles. Next subject repaired part to pressure, either by the use of hand roller or any other convenient rolling tool. If any part of the edge of the patch does not appear to be thoroughly adhered and is inclined to curl, trim same with sharp scissors."

The same authority gives full instructions for dressing a diver, and as to conduct while diving which are worth recording:

"The diver having taken off his own clothes, puts on a jersey or heavy flannel shirt, a pair of drawers carefully adjusted outside the shirt or jersey, and well secured to prevent slipping down, and then a pair of heavy stockings. If the water be cold the diver may put on two or more of each of these articles. He then puts on the woolen cap, drawing the latter well over his ears. Some divers find benefit in putting cotton saturated with oil in their ears. If the shoulder pad is used it is now put on and tied under the diver's arms. He then gets into the diving-dress, which in cold weather should be slightly warm, drawing it well up to the waist; he next puts his arms into the sleeves, an assistant opening the cuffs by means of the cuff expanders as the diver pushes his hand through the cuff.

"The diver now sits down and the inner collar of the dress is drawn well up and tied around the neck with a bit of spun yarn; the breast-plate is next put on, taking care that the rubber collar of the dress is not torn in putting it over the projecting screws of the breast-plate. The straps (four pieces) of the breast-plate are put over the bolts and secured by the thumb-screws, the center of each section being tightened first. It is usually sufficient to tighten the thumb-screws by hand, the wrench being used only when necessary. The shoes are now put on. If rubber mittens are to be used the rings are placed inside the cuffs, the mittens drawn on, the clamps being fitted into the rings and screwed tight. Next the helmet, without the front-plate, is put on; before doing this the attendant should put the helmet on his own head, and placing his mouth over the place where the air escapes, blow strongly; if in order, the safety valve will vibrate.

"A loop of the life line is placed around the diver's waist; the line brought up in front of the man's body and secured with a bit of rope passing around his neck or to the stud on the helmet. If used, the waist belt is buckled on with the knife on the left side, the end of the air-hose being passed from the front through the ring on the belt on the man's left side and up to the inlet valve of the helmet to which it is secured. The upper part of the hose is then made fast by lashing it to the eyelet on the helmet. The diver now steps on the ladder, or goes to the side, and two men are told to man the pump. The weights are put on and fastened in place; if belt weights, they are put over the shoulders, the straps being tied to the helmet and the waist straps buckled; if the horse-shoe pattern, they are hung to the breast-plate by placing the loops over the stud, the short loops going to the front, both weights being secured by a rope passing around the diver's body. When the attendant is sure that everything is right and the diver understands all the signals, he gives the word, 'Pump,' and screws in the front-plate securely; this done he takes hold of the life line and air-hose and 'pats' the top of the helmet, which is the signal for the diver to descend. With inexperienced men it is better to have a rope ladder down to the bottom, but an expert diver prefers simply a rope, but both must be weighted at the bottom.

"Each diver, while under water, requires an attendant to hold his life line and air-hose, both of which should be kept taut and clear of the gunnel, so that any movement of the diver may be felt. While the diver is under the water no talking or laughing is to be allowed.

"The diver should descend slowly, halting for a few minutes after his head is under water to satify himself everything is correct and then continue his descent. If he feels oppressed or has any humming sounds in his ears he should rise a yard or two and swallow his saliva several times. He must not continue to descend unless he feels comfortable, but return slowly to the surface. To dive to depths such as 120 to 150 feet and beyond, requires men of great practice and experience.

"On arriving at the bottom the diver should give one pull on the life line to signify that he is all right. Divers should generally walk backwards in thick water to avoid running into anything, such, for example, as iron spikes, which might break the bull's-eye. The diver takes down with him the ladder line, which he secures to the foot of ladder or rope by which he has descended. This line should be coiled up in his hand with a loop around his wrist, and as he leaves the ladder he lets the line gradually uncoil so that if he be any distance off he can find his way back to the ladder when he wishes to return. If working in thick

water, he should never let go of the ladder line when under water; if by accident he does so and cannot find the ladder, he must signal to be drawn up. He must always return to the ladder by the way he came, otherwise he may get his air-hose or life line entangled, unless in clear water where he can see plainly what he is doing. When two divers are down, they must be careful not to cross each other's paths and thus get their air-hose and life lines foul.

"In returning from moderate depths the diver should ascend very slowly, thus avoiding the effects of passing too abruptly from a considerable pressure to that of the open air, stopping now and then to become accustomed to the change of pressure. The ascent from a depth of twenty fathoms should occupy at least five minutes. It is more important to move slowly in rising than in going down, but the attendant will always lower the diver slowly."

FIG. 182a.—The Macduffee Submarine Armor.

The greatest depth on record for diving is 330 feet, which was made by using the De Pluvy suit, constructed to stand the water pressure and supplied with chemically pure uncompressed air.

The Macduffee Submarine Armor, Fig. 182a, is a similar one, made of aluminum alloy and weighing about five hundred pounds. The diver is perfectly helpless until the water pressure overcomes the weight and he can then move about like a knight in armor. There are over fifty ball-bearing joints, which leak just enough to lubricate them, and this water is pumped out by a pump run by compressed air, the pump being carried in the chamber on the back of the suit. The released air becomes available for breathing and enough more is supplied through the hose to enable the diver to carry on his work in ordinary air pressure. The arms are provided with lights and mechanical hands for carrying on work beneath the surface. Divers have been down in this suit 212 feet and were brought to the surface in eighty seconds, whereas in an ordinary suit it would have been necessary to occupy over an hour's time in ascending, to insure proper decompression.

CHAPTER XIV

REMOVING OLD PIERS

COFFER-DAMS are quite frequently constructed for the repair or removal of existing piers. A pier which was constructed in 1840 in the river Farnitz, at Stettin, Germany, became an obstruction to navigation and it was decided to remove it. The work was described in the *Engineering News* of July 14, 1892.

Its exterior showed a facing of granite laid in hard Roman cement, and soundings revealed the existence of a course of sheet-piling around the pier, with a protection of riprap at its foot. The original drawing of the pier showed a pile foundation. The specification prescribed the use of the old course of sheet-piling shown at *a*, Fig. 188, for the construction of the coffer-dam. Owing to the belief that the existing sheet-piling, after having served such a length of time, would not be sound enough to permit of its use in the erection of a coffer-dam, local contractors could not be found and the work was let to an outside contractor.

The preliminary work was begun by picking up the riprap around the foot of the pier with a claw dredger mounted on a raft. Some of the stones weighed as much as a ton. The bottom of the river, after the riprap had been cleared away, was found to be covered with a layer of concrete, consisting of pieces of brick and cement. This was brought up in large slabs. The pier itself was found to be of rubble masonry, composed of irregularly-shaped granite blocks with the interstices filled with brick, laid in cement mortar. The single stones were detached and swung off by the claws of the dredger. Their average weight was about $1\frac{1}{2}$ tons.

After the masonry had been pulled down to nearly the level of the water a row of sheet-piling, shown at *b* in Fig. 188, consisting of piles 7 inches thick, was driven to a depth of nearly 10 feet. The space between the old and new sheet-piling was filled with new clay. To keep the interior free from water two pumps were employed. After putting in the necessary bracing the work of removing the masonry to the bed of the river was continued. A shell of the latter,

however, was left standing. Then the timber platform on which the masonry had been resting and the layer of concrete below were taken out, exposing a layer of clay underneath. While attempting to pull one of the foundation piles a stream of water rushed through the opening thus formed, so that this plan had to be given up and blasting resorted to. To do this the tops of the piles were bored to a

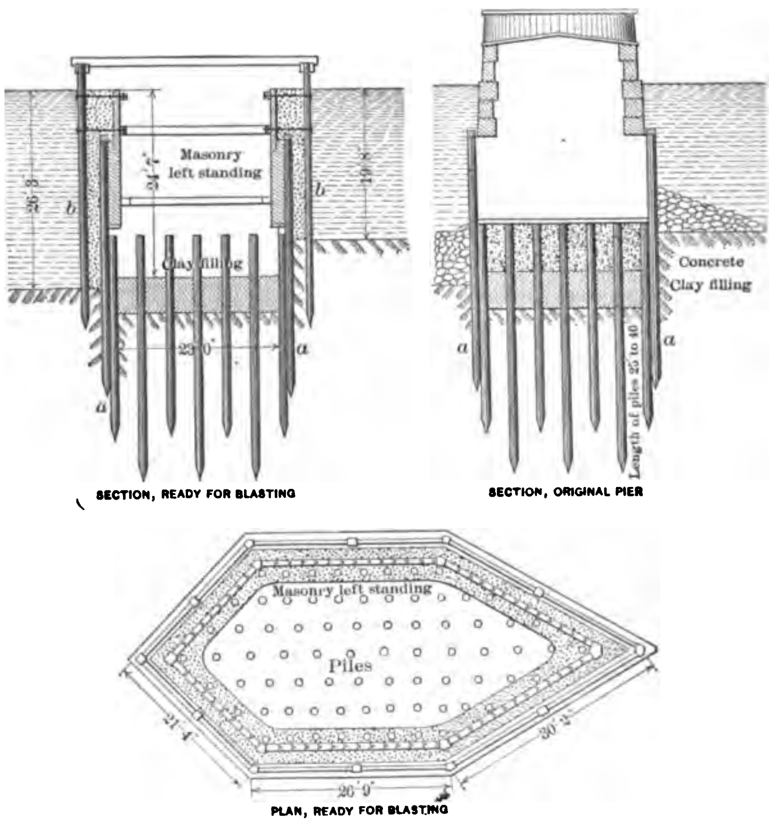


FIG. 188.—REMOVAL OF MASONRY PIER AT STETTIN, GERMANY.

depth of 13 feet and filled with 8.8 pounds of dynamite each. The initial charges consisted of 10.6 ounces in air-tight canisters. The shell of masonry left standing received four cubical charges of 8.8 pounds each. In all sixty-eight charges, consisting of 616 pounds of dynamite, were used. The electric current for the blast was divided into three currents, each being attached to an induction apparatus. The blasting, however, did not prove to be as effective as was antic-

ipated, owing to the dissolving action of the water, and several charges were taken out intact. The clearing away of the wreck was almost entirely done by the claw dredges. The piles, which were split and loosened in their sockets by the force of the explosion, were pulled up by windlasses mounted on flatboats. The work of removing the pier lasted nearly nine months and the cost was about $8700.

Another example of the removal of a pier was at Gadsden, Ala., where a pivot pier in the Coosa River had tilted. The pier had been built originally in a water-tight caisson and was supposed to have been founded on solid rock, but by some error a layer of gravel was left underneath and eventually the pier tilted down-stream 7 feet, nearly throwing the swing span into the river.

After the span had been blocked up to allow the passage of trains, a coffer-dam was built around the pier to give plenty of clearance to the old caisson. (Fig. 189.) This was constructed by driving three rows of sheet-piling through sand and gravel to bed-rock and puddling between them.

The sand and gravel over the rock were not removed from the bottom of the puddle-chamber before puddling and a great deal of trouble was experienced all through the work by leakage through the porous gravel. It is probable, too, that a poor joint was made between the sheet-piling and the rock.

Bents were erected upon the sides of the coffer-dam and by driving piles into the puddle and inside the dam, to carry a truss on each side of the span, which carried the drum and supported the main trusses at the center. When this had been tested by loading with trains of ore upon the bridge and found to be satisfactory, work was at once begun upon the removal of the old pier, by means of two fixed derricks on the false work and one floating derrick. The stones were marked as they were removed to insure their return to proper places when the pier was rebuilt, and was taken to the shore until needed again. When the masonry was all removed the grillage was broken up and taken out, after which the gravel inside the coffer-dam was cleaned out down to bed-rock. New footing courses were laid to take the place of the gravel and old grillage, and the old stonework relaid by placing each course in its former position as nearly as possible. The pier was about 80 feet high and contained about 1100 yards of masonry. The work occupied from Sept. 15 to Dec. 25, 1888, and was done under the direction of Cecil Frazer. The description is taken from the *Engineering News* of April 13, 1893.

The reconstruction of the center span of the central viaduct

across the Cuyahoga River, Cleveland, involved the elimination of the old low-level drawspan and the substitution of a high-level fixed span, as described in the *Engineering Record* of July 6, 1912. The expensive old shore-piers were retained and their foundations

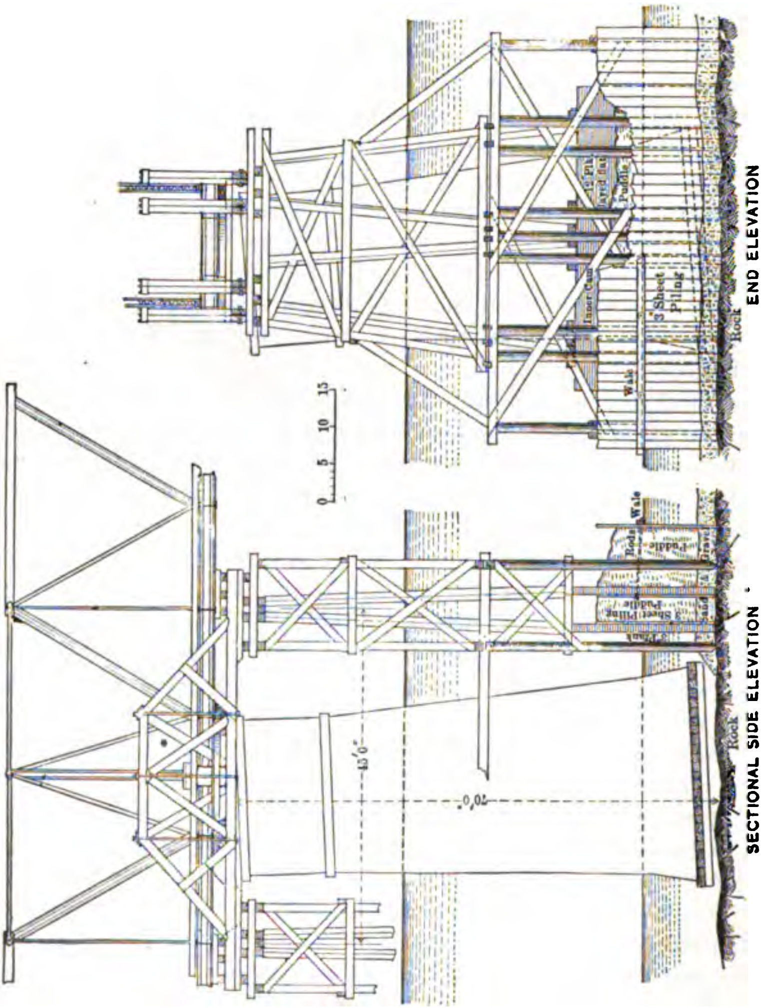


FIG. 189.—COOSA RIVER COFFER-DAM.

were reinforced and extended as described in the *Engineering Record* of June 29, 1912. The improvement also involved the removal of the old pivot pier by the Great Lakes Dredge & Dock Company, which was the contractor for the reinforcement of the old side piers. This is described in the *Engineering Record* for May 31, 1913.

The octagonal pier shaft, 54 feet high and averaging 37 feet in short diameter, consisted of thirty courses of dimension stones from 16 to 24 inches high. It rested about 4 feet below water level on an octagonal grillage, from $44\frac{1}{2}$ to $56\frac{1}{2}$ feet in short diameter, which was composed of seventeen courses of 12×12-inch timbers on a pile foundation. The stone work approximated 2190 cubic yards and weighed about 4732 tons. The grillage contained 78,000 feet board measure of white pine and 355,000 feet board measure of white oak, which together had an average weight of about 69 pounds per cubic foot, computed from the floating displacement. There were 477 foundation piles, all of which had been in place since about 1886.

The stones were wedged apart and removed by a floating derrick, very little drilling and blasting being required, so that most of the material was salvaged for future use. The stones below water level were removed in the dry by the use of the old coffer-dam constructed for the original erection of the pier, and were left in position after the completion of this work. The coffer-dam consisted of five courses of 6×12-inch timber laid edgewise around the periphery of the grillage. This was reinforced and pumped out sufficiently for the requirements of the work.

After the removal of all of the stone work the coffer-dam was again pumped dry and the buoyancy of the grillage, aided by the lifting power of the dredge, enabled the grillage to be raised clear of the foundation piles and floated. It was towed to a convenient point and two pile bents were driven as closely as possible to it, diametrically opposite each other. Three sets of $1\frac{1}{2}$-inch chains suspended from the pile caps were attached to the grillage and were jacked up to raise the latter. As the grillage was lifted the successive courses, most of which were fastened in both directions by 30-inch drift-bolts 3 to 4 feet apart, were wedged apart and pulled off by a floating derrick. This work was accomplished with considerable difficulty at first, but later on it was performed more easily and resulted in salvaging about 91 per cent of all the timber.

After the removal of the grillage from the foundation piles, the mud around the tops of the latter was dredged where necessary to a depth of 5 to 6 feet, by a clam-shell bucket and the old piles were driven down by the use of a heavy timber follower about 30 feet long, with an iron ring at the lower end, which was placed over the heads of the piles by a diver, who followed them down to an average depth of about 5 feet. None of the old foundation piles was pulled or broken off, but they were followed down, to give a clear depth of 25 feet over the top of piles. The old broken protection piles

surrounding the pier were either pulled or cut off square by divers.

The removal of the stone work was commenced Sept. 7 and completed Nov. 29, 1912. The removal of the grillage was commenced Dec. 10, 1912; the separation of its timbers was commenced Jan. 25, 1913, and was finished Feb. 25. The driving down of the old piles was commenced Dec. 16 and finished Feb. 7.

The old Eleventh Street bridge piers at Tacoma, Wn., removed by the author during the construction of a new steel bridge,

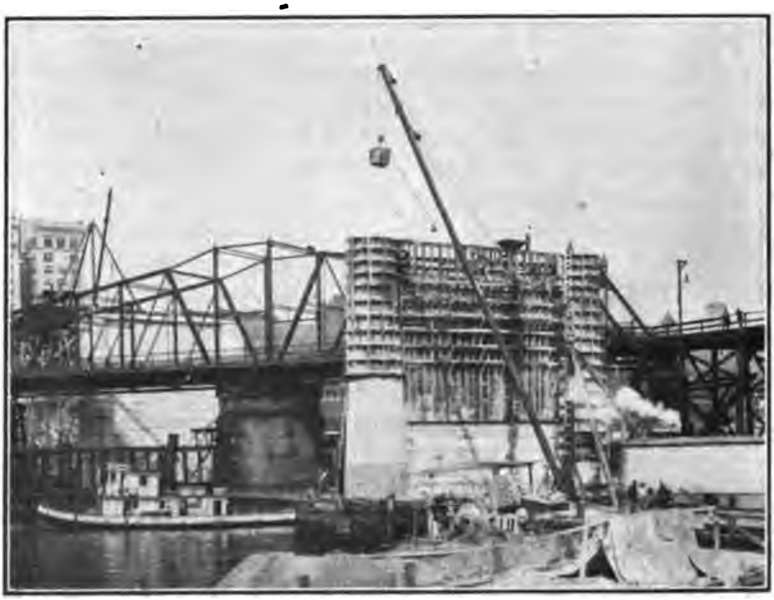


FIG. 190.—OLD PIVOT PIER, TACOMA, WN.

were steel cylinders filled with concrete and resting upon piles driven into the sand bottom. Three of the piers, 44 feet long, were made up of two cylinders, one set 8 feet in diameter and two sets 10 feet in diameter braced together with latticed struts and rods. The old pivot pier (Fig. 190), was 34 feet in diameter and 62 feet total height, the steel shell being filled solid with concrete up to within about 20 feet of the top, above which it consisted simply of a ring of concrete averaging about 5 feet in thickness to carry the circular track, while plate girders crossing at right angles at the top supported the center.

The bracing between the two cylinders forming one pier was removed, and a clam-shell bucket on one of the derrick scows used

to dig out the material around each tube, down to considerably below the bottom of the pier. Heavy lashings of 1-inch cable were placed on each cylinder a short way from the top and bottom, and then the piles holding up the tube shot off by aid of a diver, so that the tube was dropped over on its side. Each one weighed about 150 tons, so that two scows were employed to pick it up and take it out to deep water. Across these two scows were placed two fir saw-logs about 4 feet in diameter, resting on 12×12 timbers on the scows to distribute the load, as shown in Fig. 191, and then the diver picked up the lashings and made them fast around the logs at low tide. With an 18-foot rise in the tide the scows lifted the

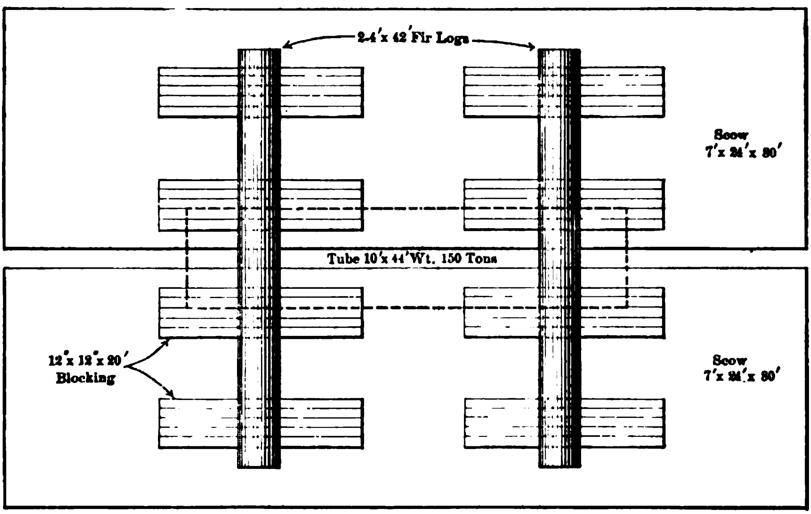


FIG. 191.—SCOWS FOR DISPOSING OF OLD PIERS.

tube clear of the bottom and it was then towed out by the tug to deep water, where the depth was about 150 to 200 feet, at a distance of a mile away from the bridge. The wire-rope lashings were let go and the tube rolled out and dropped to the bottom, the clamps having been previously removed and arranged for this purpose. The same operation was repeated with the five remaining tubes, and the crew became so expert at it that it was only a matter of a few hours to wreck one of the tubes and drop it in deep water.

The removal of the center pier shown in Fig. 192 was a more serious undertaking, and a coffer-dam would have been placed around it except for the limited channel room, consequently it was decided to drill the concrete, and break it up with dynamite so that it could

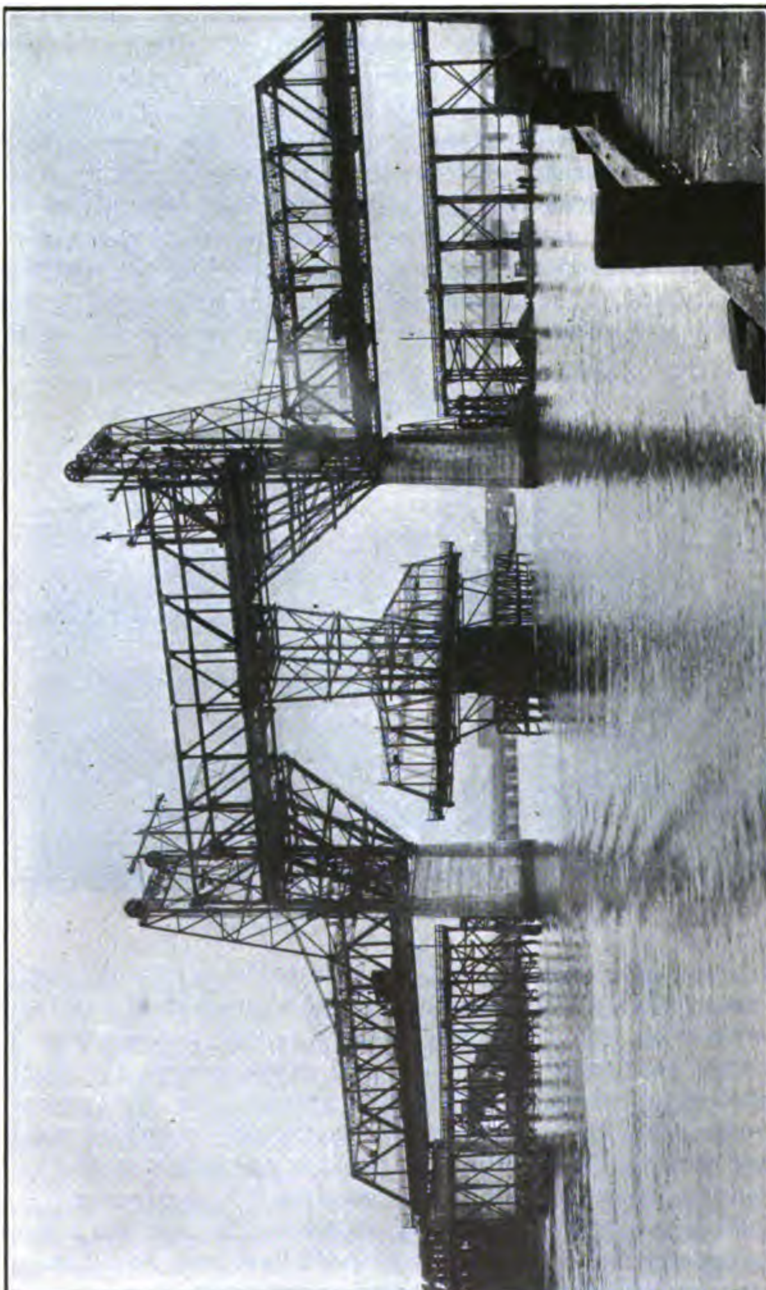


FIG. 192.—OLD TACOMA PIER DURING ERECTION OF NEW BRIDGE.

be removed with skips operated from the derrick scows. As fast as the concrete was shot down the curved plates were cut loose and removed one ring at a time, and when the pier had been demolished and removed down to about 3 feet above low water, the heads of the piles holding up the pier were reached and they were bored with a wood auger to a depth of about 12 feet. The holes were then loaded with from twenty to forty sticks of dynamite and four holes shot off at one time with the battery, breaking out a section of the pier on one side and dropping the broken concrete to the bottom, where it could be dredged up with an orange-peel bucket. Before these shots were fired the curved plates were removed clear to the bottom of the pier. After all of these shots in the piles had been fired, and the material picked up with a dredge, the divers started in and shot off the piles in rows, allowing the remaining concrete, which was very friable, to break to pieces and drop to the bottom, where it could be picked up with the orange-peel. Row after row was shot off in this way, until the entire pier was removed down to the elevation required by the United States Engineers; no requirement having been made as to the removal of the stubs of the piles, which would not interfere with future dredging, but would drop out and pop to the surface as the dredge reached them.

The work occupied about six months, the progress having been very slow on account of the fact that very small charges of dynamite were allowed, ranging from five to forty sticks at a time, although no serious jar was felt on the new bridge adjacent. Some of the concrete dropped to the bottom in chunks too large to be picked up by the dredge, and usually four or five sticks of dynamite laid on them by the diver, and discharged by the battery, broke them up in small enough pieces to be dredged out without any trouble.

The old bridge of the Oregon-Washington Railway and Navigation Company at Portland, Ore., was built about the year 1889 from the plans of Geo. S. Morison, Consulting Engineer, and was replaced in 1912, thus having lasted about twenty-three years under very severe usage and a very great increase in the train and street loading. The lower deck carried a single-track railway, while the upper deck carried the street traffic and electric double-track lines on the roadway and foot-passenger traffic on the two narrow sidewalks.

The bridge consisted (Figs. 193 and 194) of a 340-foot draw span, a 320-foot fixed span on the east side and several shorter spans on the west shore. The pivot pier was a steel cylinder 31 feet in diameter filled with concrete and resting on piles and a grillage, the piles having been cut off below low water in the usual way. The

other principal pier, at the east end of the draw span, was of cylinders 14 feet in diameter, filled with concrete and resting on piles, nineteen to each cylinder and extending up into the concrete. The main pier on the west bank was of smaller tubes but constructed in the same manner, and there were three piers still smaller, on shore at the west side of the river.

The bridge spans the Willamette River in the City of Portland, eight miles above its confluence with the Columbia. The highest water usually occurs in June and is due to high water in the Columbia. The extreme range is 28 feet. A flood frequently occurs in winter or spring from the rising of the Willamette and these floods have been known to rise 20 feet above low water, but this is an unusual

FIG. 193.—PIERS OF OLD STEEL BRIDGE, PORTLAND.

occurrence. These Willamette floods are of course accompanied by a considerable current, but during the highest stage, due to the back-water from the Columbia, there is either no current at all or else a slight current up-stream.

No drift runs except during the flood from the Willamette. These considerations making a very short season for work, the closeness of the bridge to the new structure and to structures on the bank, as well as the very frequent passing of boats, rendered the problem of removing the old piers a difficult one.

The draw span was swung around over the draw rest (Fig. 193), and blocked up while being dismantled and taken apart. The 320-foot fixed span was falseworked and removed with very little difficulty. Then the draw rest was pulled apart and the piles broken off or pulled.

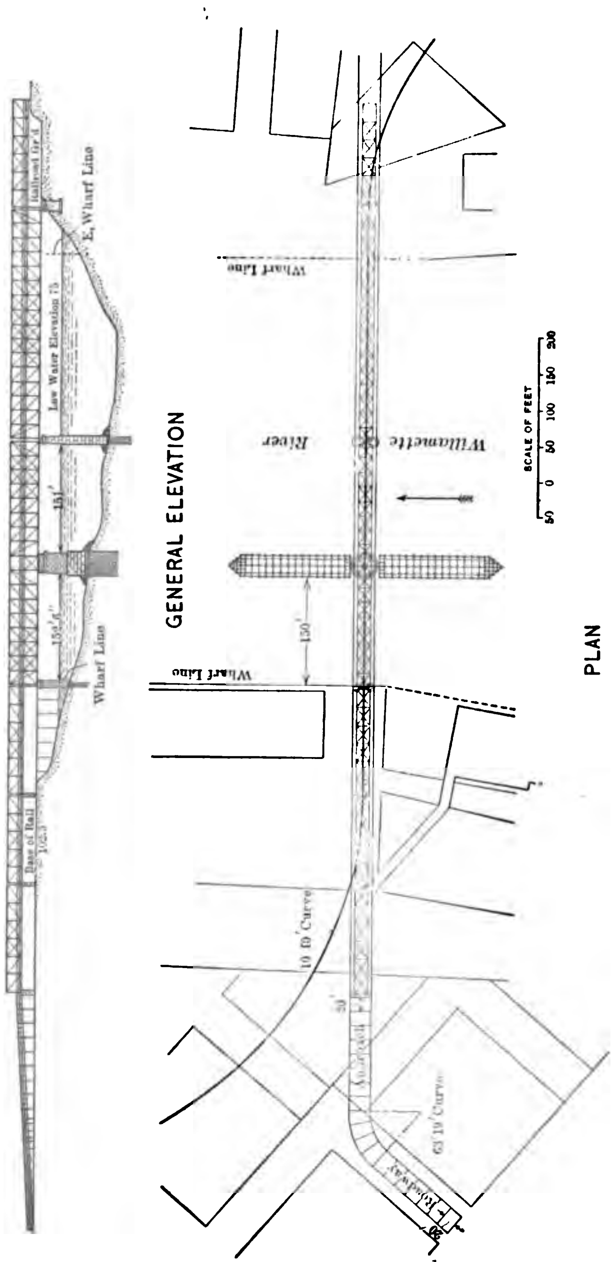


FIG. 194.—PROFILE OF STEEL BRIDGE PIERS, PORTLAND.

Owing to the fact that much of the channel had a depth (Fig. 194) of from 40 to 60 feet at low water, the Government Engineer Department allowed the piers to be blasted apart and shot off down to the depth required for navigation at low water, and the piers or material composing them deposited in the deepest places.

No particular trouble was experienced in the cutting apart of the metal work above the water, nor in removing the concrete down to that point. Owing to the very considerable depth, coffer-dams were out of the question, and it became necessary to either shoot the piers apart by the aid of a diver, and then dredge the material out into or onto scows, and dump it in deep water; or else dredge around the piers, shoot off the supporting piles, tip the tubes over into the deep water or in case they did not land in deep enough water, have the diver attach wire rope tackle to them, and drag or roll them into deep water by the aid of tugboats.

The undermining of the two cylinders composing the east pier was accomplished without any particular trouble, and by the aid of a diver the piles were shot off, so that the pier tipped over into deep water where it required no further attention. The center pier was undermined and the grillage pulled apart so that it tipped over part way into deep water, but still stood up at an angle which brought part of it above the depth required for navigation. This portion of it was shot off by a diver using charges as high as 200 pounds at one time, causing severe jars to surrounding buildings and structures. A very considerable time and about $1500 worth of dynamite were expended before it was removed to the required depth.

The pier at the west bank was allowed to remain for service as part of a wharf foundation, but the plates were cut off of the ones on shore, the concrete broken up and all hauled away by cars.

The foregoing accounts indicate that where the workmanship on piers has been first class, it costs more for labor in many cases to remove them than to construct them in the first instance. For this reason, if no other, it is incumbent upon the engineer to look ahead and so carefully plan, if possible, that the piers at least can be utilized in supporting a new superstructure when it is required.

CHAPTER XV

PUMPING AND DREDGING *

THE degree of success which has been attained in the building of a coffer-dam will be evident when the pumping process is begun. After having been pumped out, if the leakage is so small as to require only a small amount of pumping to keep it free from water, it may reasonably be considered a success.

The pumping should not exceed what can be done by a steam-siphon, a small pulsometer, or by running a centrifugal pump intermittently. Should leaks develop which cannot readily be contended with, then repairs must be made.

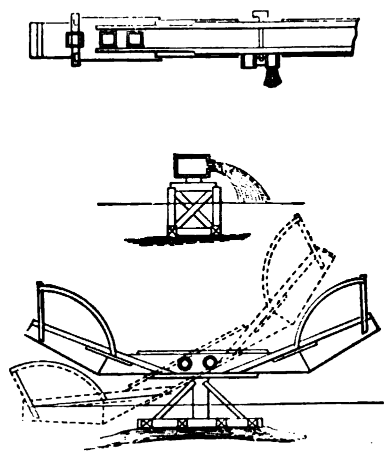


FIG. 195.—OLD BASCULE PUMP.

The use of pumps for this class of work on ancient bridges is described by Cresy. The bascule, used by Perronet at the bridge of Orleans (Fig. 195), is one of the most primitive forms. It consists of a seesaw apparatus, at each end of which ten men were placed, and 150 motions were given in it each quarter of an hour. Four cubic feet of water were raised 3 feet each time, or about 300 gallons per minute. Various other kinds of pumps were used at this bridge, among them the chaplet, which is similar to a modern chain-pump, worked by hand. Then the same device was employed, but geared to be operated by horses on a platform. A chaplet operated by a

* Attention is called to the numerous references in other chapters to the pumping plants actually employed on coffer-dams, and especially to the plant used at Topeka.
Great care should always be given to the selection of a pumping plant of the proper type and proper size, as the statements regarding capacity are often misleading. The outfit should be, if needed, one able to take care of the dredging, if the material is such that it can be pumped.

water-wheel was also used. (Figs. 196 and 197.) The large wheel had 124 cogs, while the pinion had 15, which caused the raising of over sixty-six buckets on the chain for each turn of the large wheel. At 180 turns of the wheel per hour, with each bucket lifting 290 cubic inches of water, the capacity was about 250 gallons per minute.

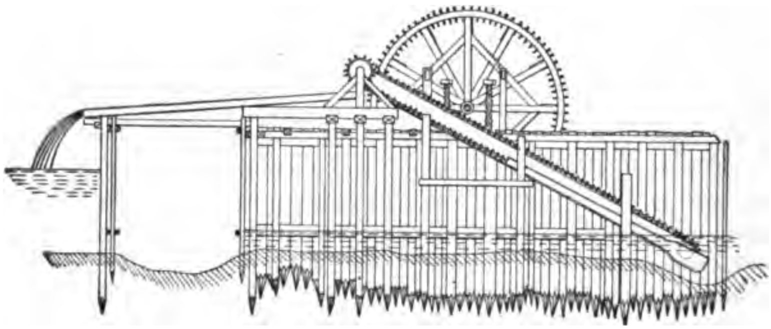


FIG. 196.—OLD CHAPELET, SIDE ELEVATION.

A great bucket-wheel was employed by the same engineer at the Neuilly bridge, 16 feet 6 inches in diameter, 4 feet 6 inches wide, with sixteen buckets.

The pumps used at the present time on very small work are usually square wooden-box lift-pumps, such as are used on large river barges, and are worked by one or more men lifting on a plunger.

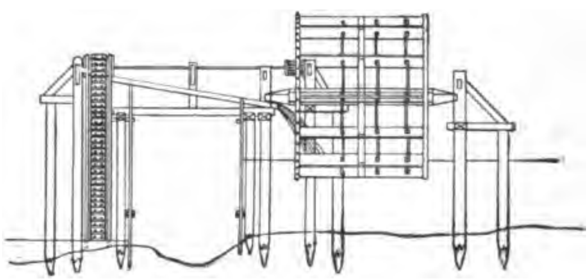


FIG. 197.—OLD CHAPELET, END ELEVATION.

These are often replaced by a similar pump of metal (Figs. 198 and 199) with a tube of galvanized metal, and often spiral-riveted. The one shown in Fig. 81 has the top and bottom soldered to the tube, while the one in Fig. 82 has screw joints. The cost of a 4-inch pump 8 feet long with fixed top and bottom would be about $6, while the screw joints would about double the cost.

Such pumps are, however, little used, as the labor becomes excessive where there is any quantity of water to deal with, and

diaphragm pumps (Fig. 200) are employed, which work on a rubber diaphragm in place of a piston and plunger, and throw a large amount of water, besides allowing the passage of sand and gravel without choking the pump. The $2\frac{1}{2}$-inch suction has a capacity of 25 gallons per minute, and the 3-inch suction of 58 gallons per minute, the list price of the two sizes being $20 and $26, respectively; the maximum lift of the pump being 30 feet.

Diaphragm pumps already described that are operated by hand are rapidly being superseded by diaphragm pumps operated by gasoline engine (Fig. 201), and they are, of course, very much more

FIG. 198.
HAND-PUMP,
SOLDERED JOINTS.

FIG. 199.
HAND-PUMP,
SCREW JOINTS.

FIG. 200.
DIAPHRAGM PUMP.

efficient and economical than similar pumps operated by hand. They should be used only where judgment dictates that they are cheaper than some type of steam pump, or else it is impossible to use a steam-driven pump, on account of the expense of moving the steam plant. The No. 3 pump, with 3-inch suction, weighs 750 pounds and has a capacity of 3000 gallons per hour, while the No. 4, with a 4-inch suction, weighs 790 pounds and has a capacity of 6000 gallons per hour. Both are operated by 3-horse-power gasoline engines.

Where steam can be obtained steam-siphons are often used, the steam being introduced into the main pipe through a nozzle, thus causing a suction, which with a 3-inch discharge Van Duzen

jet will deliver 7200 gallons of water per hour, the height of the pump above water being 11 feet, the point of discharge being 19 feet above the pump, making a total lift of 30 feet. This size will require an 18-horse-power boiler and a steam pressure of 50 pounds. The suction-pipe is 1 inch larger than the discharge, while the steam-pipe is $1\frac{1}{4}$ inches in diameter, with a jet opening of about $\frac{13}{16}$ inch.

The list price of a pump of this size (Fig. 202) is $36, the piping being extra. The pump is constructed of gun-metal and will last indefinitely. The strainer should always be used and will cost about $4 extra for the 4-inch pipe. The piping should have long bends in place of elbows where a turn is required.

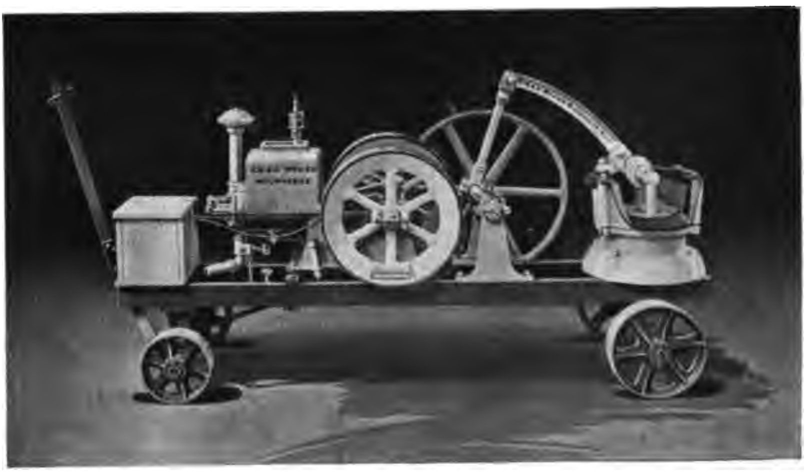
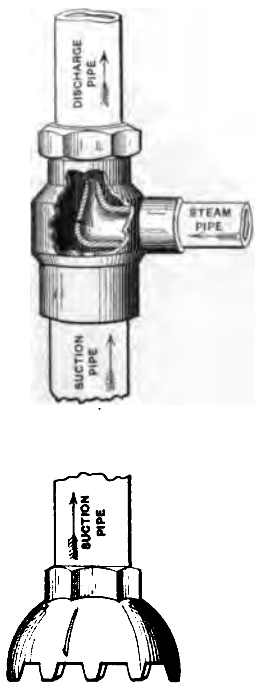


Fig. 201.—Gasoline Diaphragm Pump.

This make of pump is manufactured from $\frac{1}{2}$-inch discharge, with a capacity of 200 gallons per hour, up to 5-inch discharge with a capacity of 12,000 gallons per hour. The smaller sizes are useful for priming centrifugal pumps and for a variety of uses around a contractor's plant.

The Lansdell siphon-pump (Fig. 203) has a double suction *CC*, to which rubber suction-pipes are attached. The steam-pipe is attached to *B*, and when the steam is turned on it is blown across *A* and through *D*, thus exhausting the air from the chamber *A*. Water rises through *CC* by atmospheric pressure to fill the vacuum, and it is forced out through *D* by the steam, the velocity being proportional to the steam pressure. The steam supply should be as close to the pump as possible, to prevent condensation, and the

turns in the pipe should be easy bends, as stated regarding the Van Duzen jet. When the height to which the water is to be pumped exceeds 14 feet, the suction-pipes must be long enough to allow the center of the pump to be placed 14 feet above the water. With a 3-inch discharge, a 1½-inch steam-pipe is required and a 12-horse-power boiler. With a 6-inch discharge a 2½-inch steam-pipe is required and a 50-horse-power boiler.



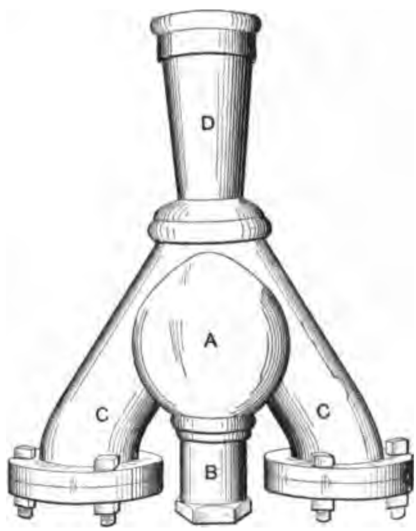


FIG. 202.—VAN DUZEN JET-PUMP.

FIG. 203.—LANSDELL'S SIPHON-PUMP.

The rated capacity of the 3-inch is 450 gallons per minute; of the 6-inch, 1800 gallons. But this would likely not be realized in practice.

The vacuum-pump which has reached the most general adoption is the pulsometer, and is in many ways better adapted to light service than a centrifugal pump of small size. There are no bearings to keep up, no belts to keep tight, and no trouble in preparing a foundation, as the pump is suspended by the hook shown in Fig. 204. The pump is operated by admitting the steam through the pipe at the

extreme top (Fig. 205), the pump having been previously primed by filling the middle chamber with water. The air-valves are closed and the steam passes into the right-hand chamber *A*, clearing it of water by forcing it into the discharge-chamber shown in dotted lines. The steam then condenses at once and the ball *C* changes its seat,

FIG. 204.—PULSOMETER STEAM-PUMP.

FIG. 205.—SECTION OF PULSOMETER.

closing the right-hand and opening the left-hand chamber to the steam. The vacuum, formed by the steam condensing in the right-hand chamber *A*, allows it to fill with water by atmospheric pressure through the suction pipe at the extreme bottom and through the chamber *D*, it being retained by the valves *E*, *E*. The steam then

enters the left-hand chamber A and the operation is repeated. The chamber J is a vacuum-chamber.

In starting the pump the steam is turned on for three or four seconds, then shut off for four or five seconds, alternating these movements until the pump is started. The steam is then turned on about half or three-quarters of a revolution, the two side air-valves opened about half a turn, and then the middle air-valve opened slowly until a regular stroke is obtained.

FIG. 206.—EMERSON PUMP.

The capacity of the 3-inch discharge, with a ½-inch steam-pipe and operated by a 9-horse-power boiler, is 180 gallons per minute when the lift is as much as 25 feet; and for the 6-inch discharge, with a 1½-inch steam-pipe and operated by a 35-horse-power boiler, 1000 gallons for the same lift.

The pulsometer is remarkably smooth in operation, and except for the slight click of the ball and the discharge of water in a steady stream, one would scarcely know it was pumping. Where a good-sized hoisting-engine boiler is in use on foundation work, it can be used to supply the steam for pumping. The work illustrated in Fig. 4 was easily kept free of water by a small pulsometer, while its use has been cited in a number of cases where the cofferdam was pumped out by a centrifugal pump, and then the leakage kept under control by a medium-sized pulsometer, which required but little attention. The pump should be provided with a strainer at the bottom of the suction-pipe, all the connections must be airtight, no sharp bends should be made in the pipe, and with dry steam successful working will result. Another pump of similar construction is the Maslin automatic vacuum-pump, which differs from it

in important details. What has been said regarding the pulsometer will apply as well to the Maslin pump.

The Emerson foundation pump is one which has been much used of late years and gives much better service in pumping out coffer-dams or cribs, than any other style of pump except the centrifugal. This pump is shown in Fig. 206 in elevation, and in section in Fig. 207. The sizes of these pumps are given in Table XXVI. In using them great care must be taken to see that the piping on the pump is properly connected up as originally received from the factory, or as may be learned from the instructions accompanying each pump. They can be swung from a derrick or hung up by sling around a timber in a coffer-dam, and as they take up such a small amount of space horizontally, they will be found very convenient, especially where the working room is limited.

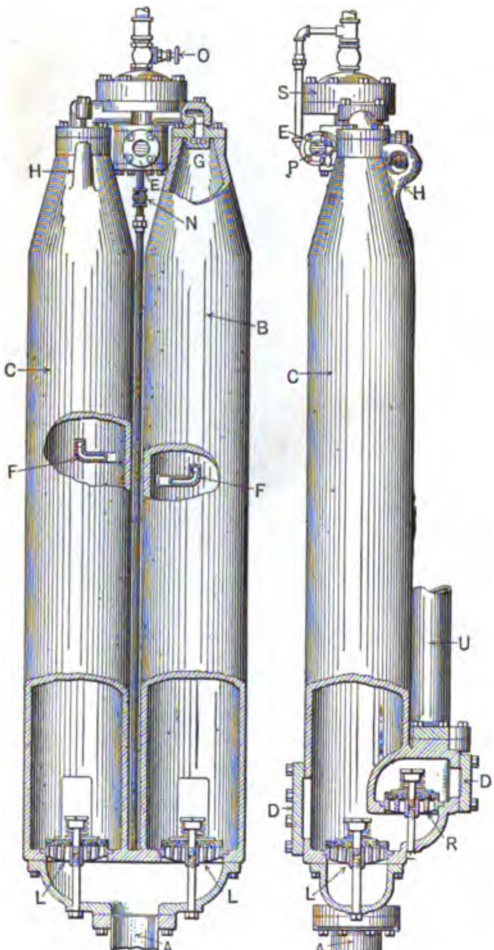


FIG. 207.—EMERSON PUMP, SECTION.

All the foregoing devices are for use where the amount of water to be handled in a given time is of limited amount, but where large quantities are to be pumped out of coffer-dams in short periods of time, recourse must be had to centrifugal pumps, which have reached a high state of perfection. Where the water is to be lifted 10 feet an ordinary reciprocating pump would exhibit an efficiency of only 30 per cent., while a centrifugal pump would have an efficiency of

TABLE XXVI.—SHOWING SIZES, CAPACITIES, WEIGHTS, PRICE LIST AND OTHER DATA OF EMERSON STANDARD STEAM PUMPS

Cipher Code Word.	Number.	Diameter of Cylinders. Inches.	Length of Cylinders. Feet.	Size of Steam Pipe. Inches.	Size of Suction. Inches.	Size of Discharge. Inches.	Capacity in Gallons per Minute.	Capacity in Gallons per Hour.	Capacity in Gallons per Day, 24 Hrs.	Dimensions over all in Inches.			Approximate Weights in Lbs.	Price List.
										Breadth.	Width.	H.		
Aid.	1	6	6	¾	3	2½	225	13,500	324,000	16½	18	97½	950	275
Ark.	2	8	6½	1	4	3	415	24,900	597,600	21½	21	104	1375	350
Alps.	3	10	7	1¼	5	4	725	43,500	1,044,000	26	24	113	1900	500
Agate.	4	12	8	1½	6	5	1200	72,000	1,728,000	29½	2[illegible]½	127	3100	700
Adam.	5	16	8	2	8	6	2100	126,000	3,029,000	43½	33	132	4400	1,150
Amos.	6	20	8	2½	10	8	3275	196,500	4,716,000	51½	36¾	135	5400	1,700

Capacities stated in table, in gallons per minute and per hour, are calculated on a head or lift of 20 feet. These diminish at the rate of about 4% for every 10 feet additional head up to 150 feet, the highest head for which we recommend our Standard Pumps.

64 per cent. For a lift of 17 feet the reciprocating type would have an efficiency of 50 per cent., while the centrifugal would reach its maximum of 69 per cent. efficiency, dropping to only 50 per cent. for a lift of 50 feet, while the other types would increase to 75 per cent. From this it will be seen that the centrifugal pump is essentially a low-lift machine.

Actual tests of pumps show that the maximum results are very seldom realized, a 9-inch discharge of one make showing an increase from 46.52 per cent. for a 12.25-foot lift to 57.57 per cent. for a 13.08-foot lift, while another make of 10-inch discharge shows a decrease from 64.5 per cent. for a 12.33-foot lift to 55.72 per cent. for a 13-foot lift. The greatest efficiency at hand is shown by a German pump with a 9¼-inch discharge, a 10.3-inch suction and a 20.5-inch disk, running at 500 revolutions. The lift was 16.46 feet and the efficiency 73.1 per cent.!

That such results are not realized on actual work is readily understood when it is considered what little care is used to properly place and operate such a plant, how little attention is paid to having a proper boiler and engine, and what lack of care there often is to keep the plant in good repair.

An ideal outfit for operating by steam is shown in Fig. 208, where the engine is directly connected to a Heald & Sisco pump, all the trouble and vexation from the use of a belt being done away with, and no loss of power through slipping of belts. The machine can be placed on the barge which carries the boiler, the suction-pipe being run horizontally across as in Fig. 208, while a short discharge-pipe discharges directly into the river. Where electric power plants are available a still better arrangement will be to have an electric motor directly connected to the pump, and all the trouble incident to the use of a boiler on the work will be avoided.

Electric power can also be used for hoisting and for pile-driving. Examples of the use of motors on hoisting machinery will be given in a later article.

The suction should always be fitted with a section of smooth-bore rubber hose (Fig. 209, *a*) to give it flexibility, a length of about 8 feet being usually sufficient. The best hose is made with a spiral metal core, which adds to its strength and durability.

The suction-pipe is ordinarily made of sections of wrought-iron pipe, with screw connections, but as this is troublesome to change sections, it will be found advantageous to use the spiral-riveted pipe with flange couplings (Fig. 209, *b*), and to have extra sections from 2 to 6 feet long, with several sections of each shorter length so that

the length of the suction-pipe can be readily changed to suit the depth of the excavation. The flanges must be provided with rubber gaskets to keep the pipe air-tight.

The strainer (Fig. 209, *c*) is used to prevent large stones, sticks, or obstructions from entering and clogging ordinary pumps, and usually comprises a foot-valve to retain a pipe full of water and make the priming easy. The strainer or end of the suction-pipe is usually

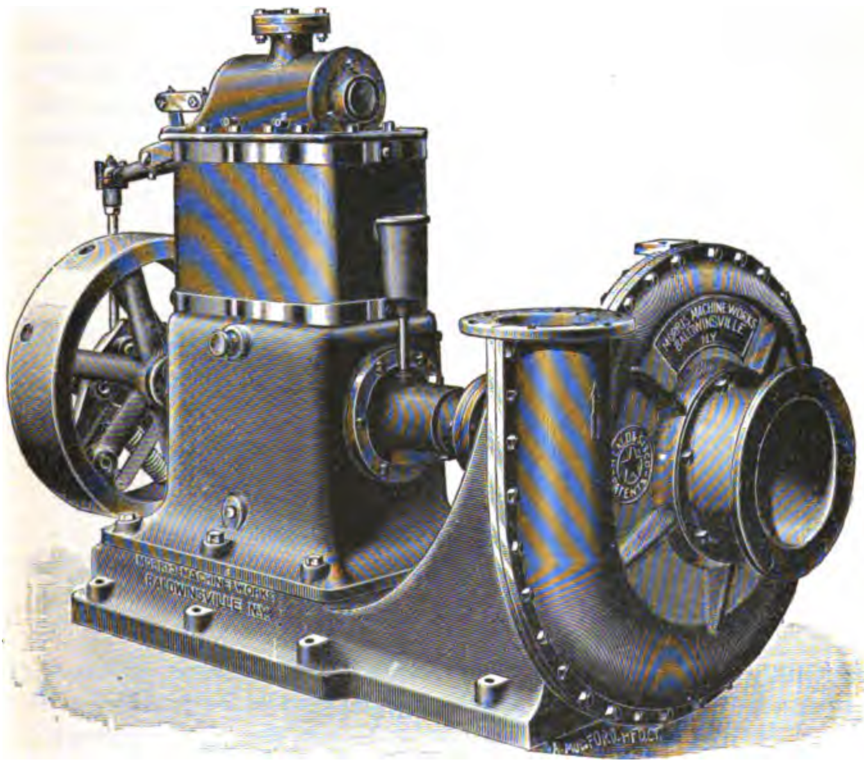


FIG. 208.—CENTRIFUGAL PUMP, DIRECTLY CONNECTED TO ENGINE.

placed in the lowest point, and sometimes a box or sump is provided, as a well into which the water is drained from the other and higher portions of the work. A small set of falls should be attached to the foot to raise the pipe and clean out the strainer when necessary.

The centrifugal pump itself must be in first-class repair to do economical work, and should be a large enough size so that it need not be run beyond its economical capacity. The style of pump to use

will depend upon the work to be done, and for coffer-dam work a vertical pump is not often used, but data regarding it is given. Where practically clean water is to be pumped an ordinary style of pump should be used, but where much mud or sand will be drawn up a sand-pump is best; and where a large part of the excavation is to be done with the pump, as at Topeka, a dredging-pump will be the proper type.

The pumping required on the Chattanooga work, 5000 gallons per minute to a height of about 15 feet, would have been done most economically by a 15-inch pump, with a 40-horse-power engine and a 50-horse-power boiler. But a pump of this size would not find ready use in a contractor's work, and for this reason two 8-inch

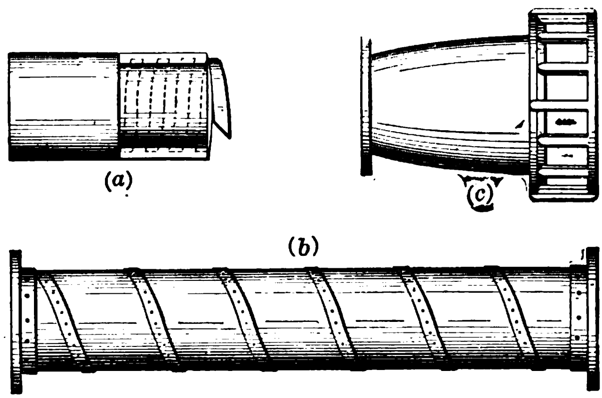


FIG. 209.—SUCTION DETAILS FOR PUMPS.

pumps would have been the better outfit to purchase, unless the work was very extensive; and each pump should be provided with a 25- or 30-horse-power engine, so as to run the pumps somewhat beyond the economical capacity, which could readily be done with a direct-connected engine, where there would be no belt to slip.

The work required on the Forth Bridge coffer-dams could also be done by the 15-inch pump above described, the lift being about 3 feet at the start and reaching 18 feet as the dam was cleared, the 340,000 gallons being pumped out in about one hour.

Centrifugal pumps are rarely required for a lift of over 20 feet on this class of work, which is only slightly beyond the economical lift, and the height should never exceed 30 feet, which would require for the 15-inch pump an engine of 75 horse-power.

The pump may be located on the coffer-dam, but in case of high

water during the progress of the work the outfit may be damaged and it is best to place the pump on a boat, as in Fig. 98, with a section of horizontal suction-pipe across to the work, which should be as short as possible.

The ordinary type of pump (Fig. 208) may be fitted with a primer, consisting of a small hand force-pump attached to one side of the pump, for filling the pump and suction-pipe. A more simple way is to provide a barrel above the pump, which can be kept full by using a small steam-jet, and, by means of a pipe with valve from the bottom of the barrel to the top of pump, the contents can be emptied into the pump to prime it. Priming may also be easily accomplished by inserting a hose into the discharge-pipe and filling the pump directly with a steam-jet.

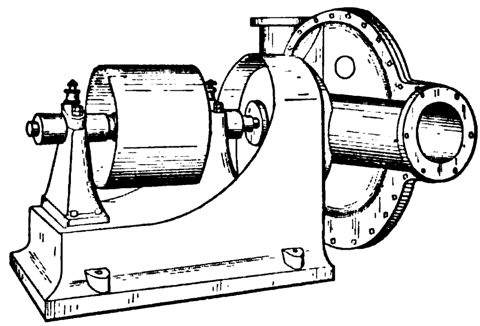


FIG. 210.—CENTRIFUGAL PUMP, DOUBLE SUCTION.

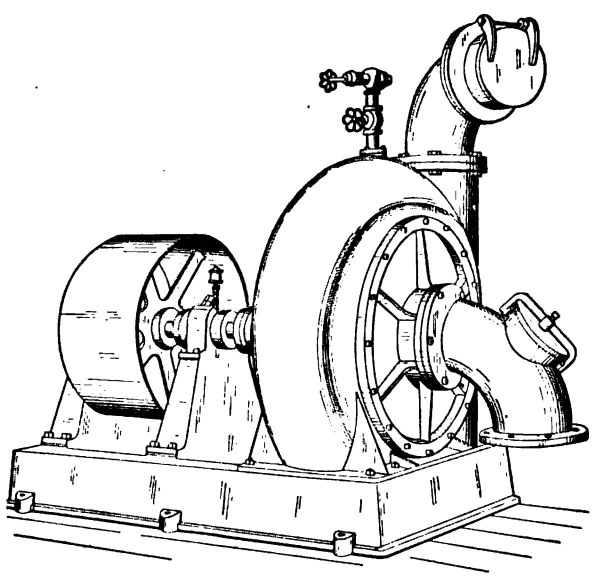


FIG. 211.—DREDGING-PUMP.

Double suction-pumps (Fig. 210) allow the water to enter on each side of the piston, and thus a perfect balance is secured, which does away with all end-thrust on the bearings. This pump is most easily primed by using an ejector, or a flap-valve such as is shown on the discharge-pipe of the dredging-pump (Fig. 211) and which serves to retain the water in the pump. Where a long discharge-pipe is to be used, a quick-closing gate-valve may be introduced into the pipe near the pump.

Where the material to be dredged out at the foundation site is mud or sand or partly gravel, it can be removed during the process of pumping by using a dredging-pump. In case there were 700 yards of material to be removed and an 8-inch pump was provided, it would not be advisable to count on more than 10 per cent of solid matter being discharged by the pump, as the suction could not be kept working close up to the sand or mud. By using a 30-horse-power engine, a discharge of 2000 gallons per minute would be reached, or with 10 per cent. of loose solid matter the excavation would be made in less than two working days.

FIG. 212.—DREDGING-PUMP PISTON.

The piston of a dredging-pump (Fig. 212) is provided with large openings to receive the material, and the one illustrated is provided with side plates so that all wear is taken off the pump-casing.

The vertical centrifugal pump, Figs. 213 and 214, is one which can be used to advantage in a great deal of coffer-dam work where it is necessary for the pump to be submerged part or all of the time. The pump can be mounted on a timber frame to be raised and lowered with a derrick. Such a pump requires no priming, and is always ready to run as soon as the engine is started. The engine or motor should be some type to be directly connected or geared on to the shaft, or else a belt from a separate engine could be used. It will very often clear a coffer-dam of water where an ordinary pump having long and possibly leaky suction will not work at all. The sizes and capacities of centrifugal pumps are given in Tables XXVII,

XXVIII, and XXIX, and the number of revolutions to run them to raise water to different heights is given in Table XXX.

The ejector as used on a centrifugal pump is best for priming; and with a foot valve on the suction, nothing is required except to turn on the steam to the ejector, and operate it until the pump and pipes are filled with water, after which when started, the pump will pick up its prime without any trouble.

One of the most remarkable pieces of work done with this class of pumps was the use of Edwards' cataract pumps in dredging the

Fig. 213.—Vertical Pump. Submerged Type.

Fig. 214.—Vertical Pump. Suction Type.

ship channel in New York Harbor. This is described in the Trans. Am. Soc. C.E., Vol. 25. The work was done by three dredges, which were much the same as small sea-going vessels, the largest being the *Reliance*, 157 feet long, and carrying 650 cubic yards of dredged material. Two separate pumps were provided, each with 18-inch suction-pipes reaching from the sides of the vessel and parallel to it down to the bottom to be dredged, being supported by suitable hoisting-tackle. These boats were kept under headway toward the dumping-ground while the dredging was in process. The average load during about a month's working of the *Reliance*

TABLE XXVII.—HEALD & SISCO STANDARD IRON HORIZONTAL CENTRIFUGAL PUMPS

No.	Capacity in Gallons per Minute.	Horse-power Required for each Foot of Lift. Min. Quantity.	Diameter and Face of Pulley in Ins.	Floor Space Required in Inches.	Shipping Weight, Pounds.	Price of Pump, Oilers and Wrench.	Price of Pump and Primer.	No.
1¼	50 to 70	.024	6×6	17×30	168	$45	$55	1¼
1½	75 to 100	.037	7×8	21×33	232	60	70	1½
2	110 to 150	.054	8×8	23×37	306	75	90	2
2½	175 to 250	.086	8×8	24×38	348	90	105	2½
3	250 to 350	.124	8×8	25×39	400	110	130	3
4	450 to 600	.223	10×10	30×40	545	130	155	4
5	750 to 900	.372	15×12	34×54	826	165	195	5
6	1,000 to 1,400	.496	15×12	37×55	965	200	240	6
8	1,700 to 2,200	.844	20×12	45×63	1,500	310	375	8
10	2,200 to 3,000	1.093	24×12	51×71	2,170	395	470	10
12	3,000 to 4,000	1.49	30×14	62×78	3,050	500	...	12
15	4,800 to 6,000	2.38	40×15	77×80	7,100	850	...	15
15*	4,800 to 6,000	2.38	30×15	60×68	3,150	710	...	15
18	7,500 to 10,000	3.73	40×15	93×103	9,000	1,300	...	18
18*	7,500 to 10,000	3.73	30×16	62×70	3,500	1,150	...	18
22	12,000 to 14,000	5.96	48×20	126×130	12,000	......	...	22

* Refers to low-lift pumps.

The number of pump is also diameter of discharge opening in inches. Where more than 25 feet of discharge-pipe is attached to pump, use one or two sizes larger than pump-discharge.

For No. 12 and larger sizes a foot-valve or flap-valve and ejector for priming is recommended.

TABLE XXVIII.—LIST OF HEALD & SISCO HYDRAULIC DREDGING- AND SAND-PUMPS

Number of Pump.	Diameter Suction and Discharge Openings, Inches.	Cubic Yards of Material they will Raise per Hour.	Horse-power Recommended for 10-foot Lift.	Diameter and Face of Pulley.	Floor Space Required, Inches.	Shipping Weight, Pounds.	Will Pass Solids, Diameter, Ins.	Price of Pump Complete, with Suction and Discharge Elbows, Flap-valve, and Ejector.	Number of Pump.
4	4	30 to 50	6	12×12	40×31	800	2	$210	4
6	6	60 to 80	12	20×12	68×40	1,700	4½	300	6
8	8	125 to 150	22	24×14	72×48	3,400	6	475	8
10	10	200 to 300	35	30×15	94×54	4,200	8	600	10
12	12	300 to 375	45	36×20	114×66	9,000	10	850	12
15	15	400 to 500	75	42×24	154×78	12,000	10	1,450	15
18	18	500 to 700	125	48×30	160×80	13,500	10	1,900	18
20	20	....	.....	......	.....	.....	....	....	20
22	22	....	.....	......	.....	.....	....	....	22

TABLE XXIX.—VERTICAL CENTRIFUGAL PUMPS

No. Pump (Diameter Discharge Opening)	Economical Capacity in Gallons per Minute.	Horse-Power Required for Each Foot Elevation.	Diameter and Face of Pulley in Inches.	Floor Space Required in Inches.	Distance Bottom of Pump to Bottom of Center of Coupling.	Coupling Bored for Connecting Shaft, Inches.	Price Extra Bearings, Each.	Price Extra Couplings Each.	Shipping Weight Pounds, Submerged Type.	Price Complete as per Foot Note, Submerged Type.	Shipping Weight, Pounds, Suction Type,	Price Complete as per Foot Note Suction Type.	No. Pump.
$1\frac{1}{2}$	70	.058	5 × 6	17 × 21	2 ft. 9 in.	1	$1.00	$1.50	110	$40	135	$62	$1\frac{1}{2}$
$1\frac{3}{4}$	90	.075	6 × 6	21 × 29	3 " 0 "	1	1.00	1.50	165	50	200	78	$1\frac{3}{4}$
2	120	.10	7 × 8	23 × 30	3 " 4 "	$1\frac{1}{16}$	1.50	2.00	198	65	250	100	2
$2\frac{1}{2}$	180	.15	7 × 8	24 × 30	3 " 4 "	$1\frac{1}{16}$	1.50	2.00	220	80	275	124	$2\frac{1}{2}$
3	260	.22	7 × 8	25 × 32	3 " 6 "	$1\frac{1}{8}$	1.50	2.00	235	95	340	147	3
4	470	.30	8 × 10	29 × 39	4 " 0 "	$1\frac{7}{16}$	2.00	2.50	380	110	495	170	4
5	735	.45	10 × 10	34 × 45	4 " 7 "	$1\frac{11}{16}$	2.50	3.00	605	140	785	216	5
6	1,050	.59	12 × 12	37 × 48	4 " 7 "	$1\frac{13}{16}$	3.00	3.50	740	170	1,050	285	6
8	2,000	1.00	18 × 12	45 × 56	5 " 5 "	2	4.00	4.00	1,320	265	1,710	445	8
10	3,000	1.52	20 × 12	51 × 68	5 " 5 "	2	4.00	4.00	1,430	330	1,925	550	10
12	4,200	2.00	24 × 14	63 × 72	6 " 0 "	$2\frac{3}{8}$	5.00	5.50	2,640	420	3,000	700	12
12*	4,200	2.00	20 × 12	49 × 62	3 " 9 "	$2\frac{5}{8}$	5.00	5.50	2,000	370	2,500	650	12*
15	7,000	3.50	30 × 16	77 × 102	6 " 6 "	$3\frac{1}{4}$	8.00	3.00	5,500	600	6,000	1,000	15
15*	7,000	3.50	30 × 15	60 × 71	6 " 6 "	$3\frac{1}{4}$	8.00	8.00	2,650	480	3,100	800	15*
18	10,000	4.50	36 × 18	98 × 126	7 " 0 "	$3\frac{3}{4}$	10.00	12.00	6,000	950	7,000	1,585	18
18*	10,000	4.50	30 × 16	66 × 78	6 " 6 "	$3\frac{1}{4}$	10.00	12.00	2,900	850	3,300	1,420	18*
20	12,000	5.40	36 × 20	73 × 92	4 " 6 "	4	15.00	20.00	4,500	1,255	5,200	2,100	20
24*	15,000	6.50	48 × 20	88 × 110	6 " 0 "	$4\frac{1}{2}$	20.00	25.00	8,000	2,000	9,000	2,800	24*

* Refers to low-lift pumps.

was 585 cubic yards and the average time of loading about forty-eight minutes, while the average number of loads per day was 6.73.

TABLE XXX.—NUMBER OF REVOLUTIONS AT WHICH PUMPS SHOULD RUN TO RAISE WATER TO DIFFERENT HEIGHTS

No.	5 Feet.	10 Feet.	15 Feet.	20 Feet.	25 Feet.	30 Feet.	35 Feet.	40 Feet.
1½	428	604	739	854	955	1045	1131	1208
1¾	348	491	601	695	777	850	920	982
2	302	426	522	603	674	737	798	852
2½	302	426	522	603	674	737	798	852
3	302	426	522	603	674	737	798	852
4	285	402	493	569	637	697	754	805
5	256	362	443	512	572	626	678	724
6	214	302	368	427	478	523	566	604
8	183	259	317	366	409	448	485	517
10	168	238	291	336	376	411	445	475
12	133	188	230	266	298	326	352	376
15	105	148	181	209	234	256	277	295
15*	151	213	261	301	337	369	399	426
18	105	148	181	209	234	256	277	295
18*	151	213	261	301	337	369	399	426

Above table gives *correct* speed of pumps as employed under usual conditions of pumping. If water must be forced through a number of bends and elbows, or a great length of piping, the above speed must be somewhat increased.

Use large pipes and easy bends wherever practicable, as they save power.

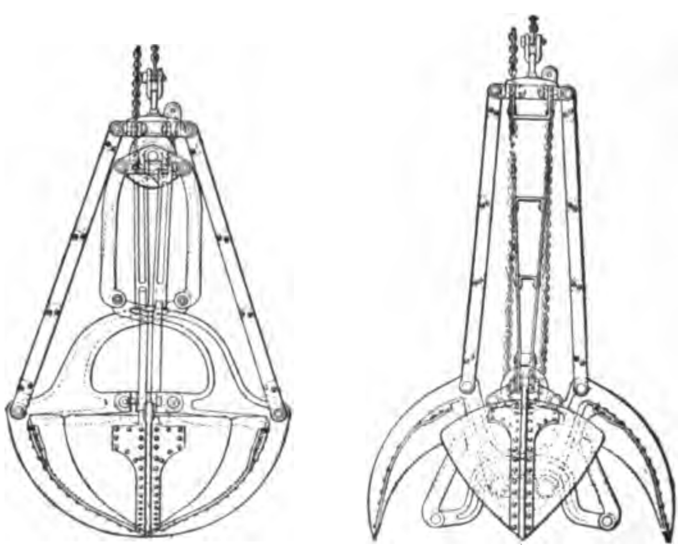


FIG. 215.—LANCASTER GRAPPLE.

These dredges removed the enormous quantity of 4,299,858 cubic yards of material at an average price of 24.48 cents per yard, the lowest price being about 17 cents, the average price paid for other forms of dredging being 40.53 cents. On foundation work the amounts to be removed would be small and the cost for this reason much higher, yet owing to the smaller cost of the plant that

FIG. 216.—RICKARDS ORANGE PEEL BUCKET.

would be required the cost need not be greatly in excess of the above. It is usual, however, as the amount to be dredged will cost such a small proportion of the total cost of the substructure, to figure from $1 to $2 per yard for excavation in ordinary coffer-dams.

Reference has already been made to hand dredging, and a very cheap and effective scraper was illustrated in Fig. 11. Where

dredging is to be done in tubes, wells, or puddle-chambers, it can be done by a clam-shell dredge or grapple such as was shown in Fig. 57, in use on the Hawkesbury foundations.

The Lancaster bucket (Fig. 215) is a well-known form of this type of machine, and can be operated from an ordinary derrick which is served by a double-drum hoisting-engine. This dredge

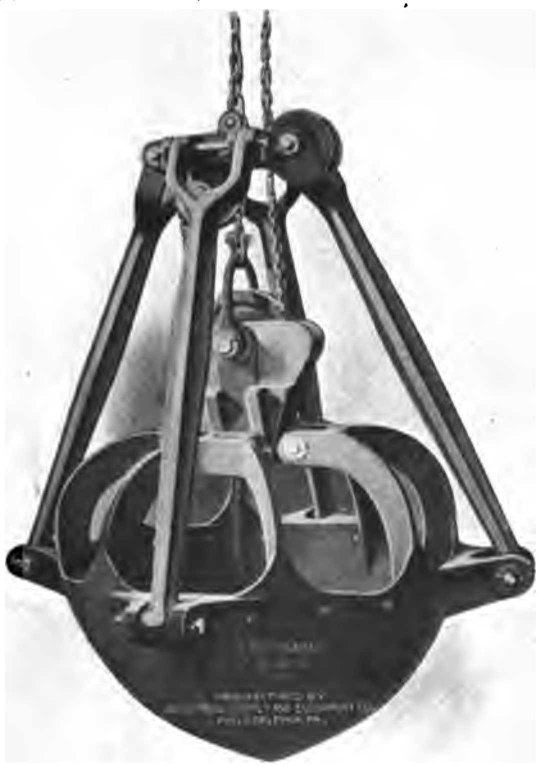


FIG. 216a.—RICKARDS ORANGE PEEL BUCKET.

will work best, of course, where there is some depth of soft material to be removed. While a large dredge would generally be hired by a contractor, these buckets can be owned by him and the work carried on cheaply and conveniently.

The Rickards cast-steel orange-peel buckets are of a type that is first-class for use where there is more or less loose rock to pick up, and where extra strength is required in the blades and arms.

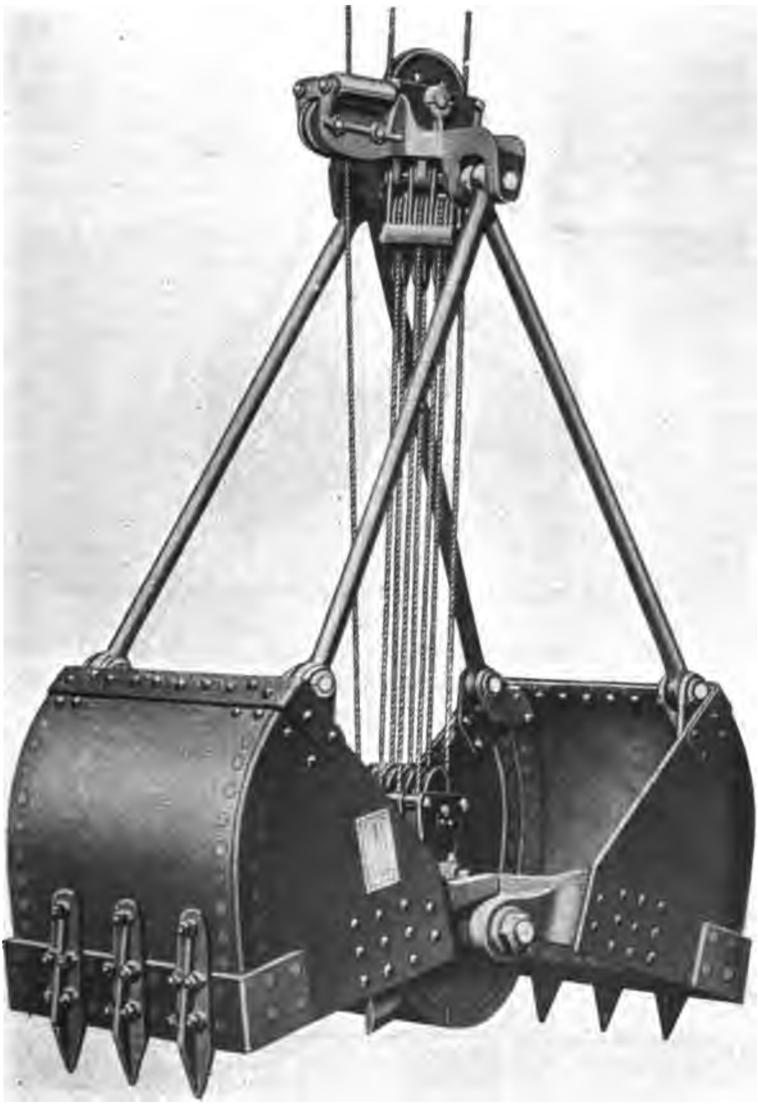


FIG. 217.—OWEN CLAM SHELL BUCKET.

This bucket is shown in Fig. 216, and the sizes, weights, and prices are given in Table XXXI. Where the material is of a uniform

FIG. 218.—WILLIAMS HERCULES FOUR-PART CLAM SHELL BUCKET.

nature, running from mud and sand to stiff clay or gravel, some of the clam-shell buckets that have extra interior tackle or levers for

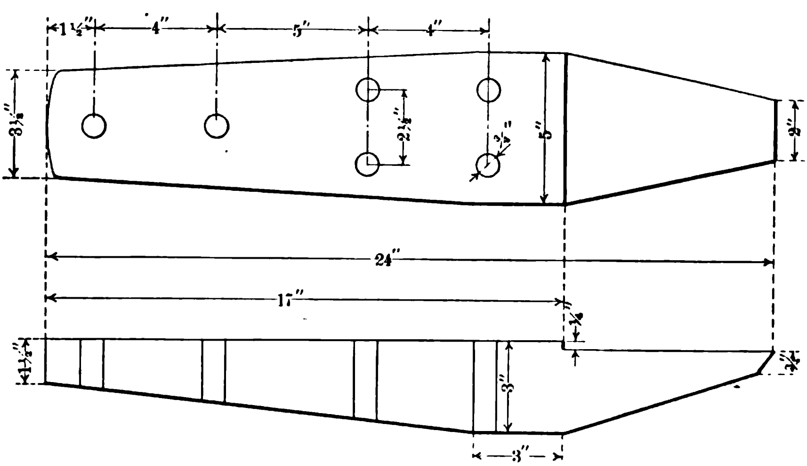


FIG. 219.—CLAM SHELL BUCKET TOOTH.

giving a powerful closing force, to close the bucket before it starts to lift off from the bottom, are best used. Some of these buckets

will do excellent work in very hard clay and packed gravel, but will not work very satisfactorily in hardpan and cemented gravel, except for the larger sizes which have weight enough to make them take hold.

The Owen bucket, Table XXXII, is one of this type, having a seven-part tackle between the top and center shaft, as shown in Fig. 217, to close the bucket. Should it be desired to use this in extra hard material, it should be built of special extra heavy design with extra large pins and bearings, and extra large riveted connections everywhere.

TABLE XXXI.—RICKARDS CAST-STEEL ORANGE-PEEL BUCKETS

Capacity.	Approx. Weight in Pounds.	Price.	Capacity.	Approx. Weight in Pounds.	Price.
1 cu. ft.	125	$175	1¼ cu. yds.	4300	$1200
2½ "	450	250	1⅓ "	4800	1350
5 "	800	450	1½ "	7000	1600
7 "	900	475	2 "	8000	1750
9 "	1000	500	2½ "	9000	1950
12 "	1600	725	3 "	10000	2150
15 "	1800	775	4 "	15000	3600
21 "	2400	1000	5 "	17000	4000
1 cu. yd.	4000	1050	6 "	20000	4400
32 cu. ft.	4100	1150	8 "	23000	5200
			10 "	27000	6000

TABLE XXXII.—OWEN BUCKETS

DATA ON TYPE "G" ORDINARY WEIGHT

Capacity Cu. Yds.	Weight.			Extra Jackets, Pounds.	Open.				Width. Ft. In.		Closed.			
	Bucket Less Jackets, Pounds.	Inside Jackets, Pounds.	Outside Jackets, Pounds.		Height. Ft.	In.	Length. Ft.	In.	Ft.	In.	Height. Ft.	In.	Length. Ft.	In.
½	2575	200	225	300	7	0	5	2	3	0	6	0	4	0
¾	2975	200	225	300	8	2	5	10	3	0	6	10	5	0
1	3175	200	225	300	8	10	6	6	3	0	7	2	5	8
1¼	3575	200	225	300	9	8	7	3	3	0	7	6	6	6
1½	4475	250	325	420	9	6	7	3	4	0	7	10	5	10
2	5425	250	325	420	10	8	8	2	4	0	8	7	6	10
2½	6525	250	325	420	11	6	9	0	4	0	8	8	8	0
3	7650	275	375	480	11	10	9	6	4	6	9	0	8	6
DATA ON TYPE "H" EXTRA HEAVY														
1½	5275	250	325	420	0	6	7	3	4	0	7	10	5	10
2	5795	250	325	420	10	8	8	2	4	0	8	7	6	10
2½	6625	250	325	420	11	6	9	0	4	0	8	8	8	10
3	7650	275	375	480	11	10	9	6	4	6	9	0	8	6

The Williams buckets (Fig. 218), especially of the Hercules type, are also very satisfactory, but the same criticism should be applied to them in regard to being built extra heavy for hard digging.

The teeth that come on any of these buckets from the factory are practically of no use at all, and should be replaced in the first instance with heavier and better designed steel teeth. The design shown in Fig. 219 is one which has given excellent service. The use of these buckets will be further described in Chapter XVI, under the discussion of clam-shell dredging.

FIG. 220.—ELEVATOR SAND-DIGGER.

Sand-diggers such as were mentioned in Chapter II can often be hired where other means are not at hand, or they can be rigged up very cheaply if necessary. A smaller one than Fig. 220 can be built on a common barge, the engine being an ordinary one with a vertical boiler, while the buckets are mounted in a very simple manner and operated through a well in the center of the boat. Such a dredge will dig about 100 yards of sand per day, with only two men to attend it, and will use less than one-half ton of cheap coal, the total cost per yard thus running below five cents. Large elevator dredges of this type are very elaborate affairs, and as they are in wide use they can often be hired for making excavations.

CHAPTER XVI

CLAM-SHELL DREDGES, DRILL SCOWS AND ROCK BREAKERS

THE use of clam-shell buckets such as have been described in the preceding chapters has been developed almost entirely as a matter of practical experience. The ordinary orange-peel or clam-shell closes directly and quickly, and opens to discharge its load in the same manner, so that they can be operated and the boom swung as well by the hoisting and closing lines, which are splayed out near the foot of the boom, or on a frame just back of that point.

Most small orange-peel or clam-shell rigs are fitted with a swinging engine and bull-wheel to swing the boom, and the hoisting and closing lines run directly up from the bottom of the boom or mast, as on an ordinary derrick. On account of the slow closing and opening of the Owen and Williams buckets, and others of that type, it is necessary that separate swinging machinery be provided.

Orange-peel or clam-shell buckets can be operated from ordinary derrick scows by the usual double-drum engine, but if it is desired to have a live boom, a 3-drum engine must be used to handle the two lines to the bucket and the other line to the topping falls, or else, if a double-drum engine is used, an idle drum, Fig. 221, must be employed to take up the closing line from the boom. This idle drum is operated by a separate wire rope, running to a counterweight from the short barrel at one end of the drum. These rigs can be operated by any good hoisting engine of sufficient size, using good judgment about the speed of operation, and about keeping the cut properly cleaned up in front of the machine.

The clam-shell dredge shown in Fig. 222, designed and built by the author, is very similar to the design shown in the detail drawing in Fig. 223, with the exception that the latter has a hull 40 feet in width, instead of 30 feet, as in the photograph. This additional width is necessary to prevent the machine from listing badly when the bucket is fully loaded and swung around to the side. The machine shown in detail is designed to carry a $2\frac{1}{2}$-yard bucket, weighing with the load about 10 tons, which must be hoisted on a

single line, and will require a double 10×15 double-drum main engine for proper operation. The slewing or swinging engine should be a double 5×8, the front spud engine a double-drum double 7×10, and the rear spud engine a single-drum double 7×10. The dredge should have a direct-connected electric-light plant large enough to operate one hundred 16-c.p. lamps, including lamps in a 7-cluster 350-c.p. light on the boom, or, where necessary, this can be supplemented by a 1500-c.p. Milburn light near the foot of the boom.

FIG. 221.—IDLE OR BUCKET HOLDING DRUM.

The boiler for this machine should be of locomotive or internal fired type (Fig. 76), or possibly a Scotch marine boiler as described in Chapter XIX, of about 200 horse-power. The machinery should be arranged to operate by hand levers entirely from the pilot house. The hull of the machine is 40×90×8 feet deep, and has 8-inch side strakes; 2 solid 6-inch longitudinal bulkheads, 6×12 keelsons and deck beams, while the planking is 4×12 on the bottom and 3×9 decking. The bottom and sides are calked with 4 threads of oakum well hawsed in, and the seams filled with cement up to the water-line. The deck is calked with 3 threads of oakum, and the seams on the deck and on the sides above the water-line thoroughly pitched. The bottom and sides up to the water-line are painted with two coats of copper paint, and are then covered with tarred ship's felt and

1-inch sheeting, after which two more coats of copper paint are applied. The hull is braced transversely as shown, and is thoroughly bolted together with clinch-bolts, and in addition to clinch-bolts, the side planking is edge-bolted with $\frac{3}{4} \times 20$ drift-bolts every 4 feet, staggered. The spud casing and chocks are through-bolted with screw-bolts, and the spud masts fastened on in the same way. The forward spuds, $24 \times 24 \times 60$ feet long, and the rear spud, $20 \times 20 \times 65$ feet long, have cast and wrought points, respectively, as shown, and are strapped lengthwise with $4 \times \frac{3}{4}$ straps to strengthen them, set into each side and drift-bolted to the spuds. The fittings of the mast,

FIG. 222.—CLAM-SHELL DREDGE.

boom and sheer legs are all of cast steel, and designed with a high factor of safety, so as to cause no trouble from breakdowns. The 20-inch sheaves at the foot of the mast and point of the boom have 5-inch pins and 4-inch hubs, so as to avoid undue wear. Such a dredge as this will in soft material dig a bucket full every fifty seconds, and should average sixteen hours running time out of twenty-four.

The crew for double shift consists of a captain, 2 levermen, 2 engineers, 2 deckhands, 2 scow men, 1 blacksmith, 1 helper, and will use from 2 to 3 tons of coal per day. Such a dredge should have

two 250-yard dump scows, and at least a 50 horse-power gasoline tug to tow the scows and carry supplies. The bucket used should be of heavy construction, and would weigh for the $2\frac{1}{2}$-yard size between 4 and 5 tons. The weight of various sizes of these buckets is given in Table XXXII. The dredge with dump scow is shown in Fig. 224, the dump scow being designed with center wooden rollers to wind up the closing chains. These rollers should be of hard wood, as Douglas fir or Southern pine will not stand the strain

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FIG. 224.—CLAM-SHELL DREDGE WITH DUMP SCOWS.

of continuous heavy loads. The usual machinery for closing dump scows consists of a steel shaft on one side, on which to wind the chains. Temporary pockets may be built on a flat scow as shown in Fig. 222. The cost of such a dredge is about $35,000, and the scows about $4500 each. Machines of light construction for temporary use can be constructed for about $12,000.

The type of dredge bucket in use in California, and known as the "Stockton" bucket, is shown in Fig. 225. This bucket is closed by heavy curved arms, and the bowls of the bucket are made of cast

steel. The closing line is attached to the arms, while the opening line is attached to the bowl by lugs and shackles as shown. The

FIG. 225.—"STOCKTON" CLAM-SHELL BUCKET.

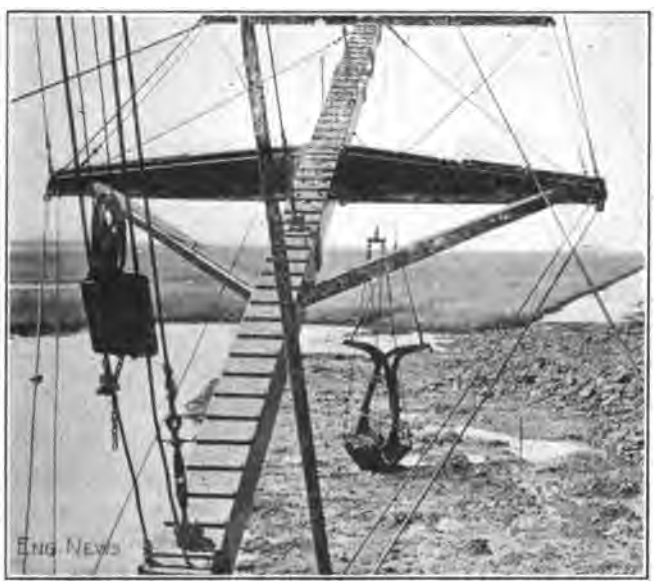


FIG. 226.—"STOCKTON" CLAM-SHELL BOOM.

trussing of the boom, and the bucket closed are shown in Fig. 226, while one of these dredges with a 150-foot boom is shown in Fig.

227, which also shows plainly the frame on the sheer legs, which carries the sheaves to swing the boom with the bucket lines. The spud frame can also be seen, for picking up the spuds through spud wells inside the hull.

FIG. 227.—CLAM-SHELL DREDGE, "STOCKTON" TYPE.

The hulls of these machines are made with a beam of from 50 to 60 feet, and a length of from 100 to 130 feet, with the depth usually about 10 feet. The buckets, ordinarily, are of a size that handle from $3\frac{1}{2}$ to 5 yards for each bucketful, but many of the larger dredges of this type have buckets ranging in size from 6 to 15 yards capacity. Such a dredge should show in actual operation about sixteen hours'

running time of twenty-four, and many of the California levee contracts are taken in the neighborhood of 5 cents per cubic yard, which will cover a fair profit to the contractor, where the digging is easy and the yardage large. These machines are sometimes used for loading material into dump scows, but are not nearly so satisfactory for this purpose as machines which are swung with a bull-wheel.

The engineer using a clam-shell dredge on foundation work must bear in mind that under such conditions no large yardage can be handled per day, as can be done by a dredge on regular dredging work. In dredging over a considerable area, before bridge caissons are located, the dredging may be done as low as 40 cents or 50 cents per cubic yard, where the material can be easily dug, but if work is to be done inside the caisson for sinking it, it will seldom run less than 75 cents per cubic yard, and on small work the excavation should be figured at approximately $2 per cubic yard for medium hard digging. The comparative amounts that can be dug in different kinds of material are shown in Table XXXIV.

Where the bottom is soft or shelly rock that has to be removed before placing the foundation for a pier, a dam, or other engineering work, it very often becomes necessary to drill and shoot it before trying to dredge it out with either a dipper or clam-shell rig. Some work of this character, where the material was afterward dredged out by a dipper dredge, was done under J. E. Hall, Assistant United States Engineer, at Tuscumbia Bar, Tennessee River. The description given in Professional Memoirs is quoted in full, as the equipment was of a type that can be easily and cheaply rigged up for any work of this character, at small expense, and by using ordinary rock drills, boilers and other equipment easily procured.

"This obstruction to navigation is 211 miles below Chattanooga and 253 miles above the confluence of the Tennessee River with the Ohio, at Paducah. It begins at a point about 3 miles below the Florence bridge, abreast of the Sheffield power plant and extends approximately 2 miles down the river.

"This shoal is composed of a series of blue flint limestone ledges, which are overlaid with a thin coating of gravel. The ledges are about level transversely, but have a considerable down stream dip.

"The project for improving this place contemplates excavating a channel near the south shore, 150 feet wide and 5 feet deep at extreme low water, from deep water above to deep water below the bar. The material to be moved being very hard flint rock, it was necessary to drill and blast it before it could be dredged.

"It was originally intended to contract this work provided a favorable offer was received, and it was not until after the proposals were opened about June 1, 1911, that the decision was made to do the work with hired labor.

"The work here being the first rock excavation where drilling and blasting was necessary, on this section of the river, we were at an experimental stage in

regard to plant and methods. A small amount of drilling had been done on the upper division (above Chattanooga) where rafts were used for carrying the drills, and a derrick boat was used with each raft for a tender, the derrick being necessary in moving since there was not enough buoyancy in the solid timbers of the rafts to float the drills when the spuds were raised. Extensive drilling operations being necessary here in rock where the depth of water varied between a few inches and 5 feet at low water, it was thought advisable to design floats with sufficient buoyancy to carry the drills, men and material necessary to operate them, and to also keep them as light as possible in draft, as it was necessary to pass them over some very shallow water.

"When it was definitely decided that this work would be done by the United States, requisitions were promptly submitted for drills, material, lumber, etc.,

FIG. 228.—DRILL PLANT, TENNESSEE RIVER, ALSO CHANNEL AFTER BLASTING.

necessary for building the floats. The first shipment of ten drills was received August 15, and a second lot September 20. These were put in commission as quickly as practicable after being received, and on September 21 three drill units were in operation, each mounting six drills.

"The floats used for the first season's drilling (1911) were composed of of nine small boats, the dimensions of each being: depth 1 foot; width, 5 feet, and length, 25 feet. These were arranged in three rows with three boats in each row and a space of 2 feet was left between the lines of boats through which to operate the drills. During the first season they were turned longitudinally across the current and the float held in place with spuds which were 6 inches square. The dimensions of the floats when the boats were so arranged were 19 by 75 feet. At each placing of the drill unit two lines of holes were drilled extending one-half the way across the channel and twelve holes put down in each line. These lines being 7 feet apart made the average spacing of the holes

6½ feet by 7 feet, and the holes were put down to a depth of 7 feet below low water.

"The drilling was carried on on this basis until December 15, 1911, when it was terminated by high water and bad weather. During this season's work about 2,500 linear feet of channel, beginning at the upper end of the work, was drilled and blasted. This was over the lightest part of the drilling, as it began at grade point and the depth decreased very gradually, so that the depth of the holes varied from 2 to 6 feet in the rock.

"Subsequent dredging has proven that this spacing of the holes was too wide for this class of material and not deep enough for properly loosening it up. Where the detonation was perfect the rock was usually broken up, but often in such large pieces that they were very difficult to handle, and it was sometimes necessary to break them again with mud-capped shots before they could be handled, but when any of the holes failed to fire an unbroken area was left which was necessary to redrill. The best results both in drilling and blasting were gotten at a low stage of the river, the effect of a rise being noticeable when a 2-foot stage was reached. The increased current due to the rise made it hard to hold the floats in place and the difficulty of detonation was also increased, as the current and drift would often break the connections.

"The following table shows this season's work, giving the unit cost of drilling for each month, also the stage of the river, depth of water over the rock drilled and the depth of the drilling in the rock.

Date. 1911.	Linear Feet.	Average Depth of Holes. Feet.	Gauge. Feet.	Depth of Water over Rock. Feet.	Cost.	Unit Cost.
August	3,520	4 to 5	.4 to 2	3 to 5	$1,336.60	$0.38
September	6,338	3 to 5	.3 to 1.5	3 to 4	2,871.20	.40
October	6,921	3 to 5	1 to 5	3 to 7	3,450.00	.50
November	10,373	4 to 5	1 to 3	2 to 6	4,634.39	.43
December	1,679	4 to 6	2 to 12	4 to 7	1,007.40	.66
Total	28,831				$13,299.59	$0.46

Average unit cost for season 1911, $0.46. Number of linear feet of channel drilled, 2,500.

"The small progress made in drilling during the month of August was due to the late start and limited number of drills, which also affected the work in September. The October work was also retarded by a rise which caused a week's suspension. There was only a few days in December when the river was low enough for good results, but the force engaged in drilling was held together until the 15th, on which date they were disbanded owing to high water and bad weather.

"During the winter and spring dredging was carried on over the drilled area, affording an excellent opportunity to study the spacing of the drilling and the depth. A great many small areas of unbroken material were found which were probably due to the failure of charges to fire, but in some instances where the detonation seemed complete points were left between the holes which the dredges could not reduce to grade. These unbroken areas and high points showed conclusively that the spacing of the drilling was too wide and holes too shallow.

"Extensive repairs being necessary to the floats before entering on another season's work, it was decided to rearrange the boats composing them, turning them lengthwise with the current and leave the opening between the boats 1 foot wide. The floats used in the season of 1912 were composed of thirteen boats, dimensions 1 by 5 by 25 feet long, put together in this way making the spacing of the lines 6 feet apart instead of 7 as used the previous year. The dimensions of this float were 25 by 77 feet. (See accompanying illustrations, which give a good idea of their construction and also show the appearance of a section of channel after being blasted.)

"At each placing of the drill unit or float, twelve lines of holes were drilled, each line having six holes. As the length of the boat was 25 feet, the spacing of the holes was made 4 by 6 feet and the holes put down to a depth of 9 feet below low water. In addition to the advantage of narrowing up the spacing, boats being placed lengthways with the current was an advantage, in that they were less affected by the action of the current and drilling could be carried on at a higher stage of water.

"Excessive rainfall during the spring prevented our resuming the drilling until June 15, and very materially interfered with the progress during July. After July, the work was continued without interruption until the latter part of December, when it was terminated for the season by high water.

"The following table shows the season's work during the season of 1912, giving unit cost for each month, stage of river, depth of water over rock, and depth of holes drilled:

Date. 1912.	Linear Feet.	Average Depth of Holes. Feet.	Gage. Feet.	Depth of Water over Rock. Feet.	Cost.	Unit Cost.
June........	3,720	7 to 8	3 to 5	3 to 5	$1,523.19	$0.41
July........	3,256	7 to 8	3 to 6	3 to 6	1,465.00	.45
August......	15,120	7 to 8	2 to 3	2 to 4	5,727.28	.38
September...	15,752	6 to 7	1 to 3	1 to 5	6,301.85	.40
October.....	20,250	5 to 7	1 to 2	2 to 4	9,317.50	.46
November . .	20,042	5 to 7	.5 to 1.5	3 to 4	10,462.89	.52
December...	7,568	5 to 7	1 to 6	3 to 7	6,659.84	.88
Total.....	85,708	....	....	....	$41,457.55	

Average unit cost for season, 1912, $0.483. Number of linear feet of channel drilled, 2500.

"In comparing this season's work with that of 1911, it will be noted that the unit cost of the drilling is 2 3-10 cents in excess of the 1911 cost. This may be due to considerable advantage in weather and river conditions in 1911, and also to the fact that the drills were all new in 1911 while in 1912, especially near the close of the season's work, a number of them were badly worn and would not deliver a normal stroke. After September the cost is also augmented by the shortening of the days, making it necessary to work a longer number of hours at night. While the plants are very well lighted, quite a falling off is noted in their progress when comparing the result of an hour's work at night with an hour's work in the day. Early in October a number of the best drill men left the work to go back to their old places at the furnace, and it was nec-

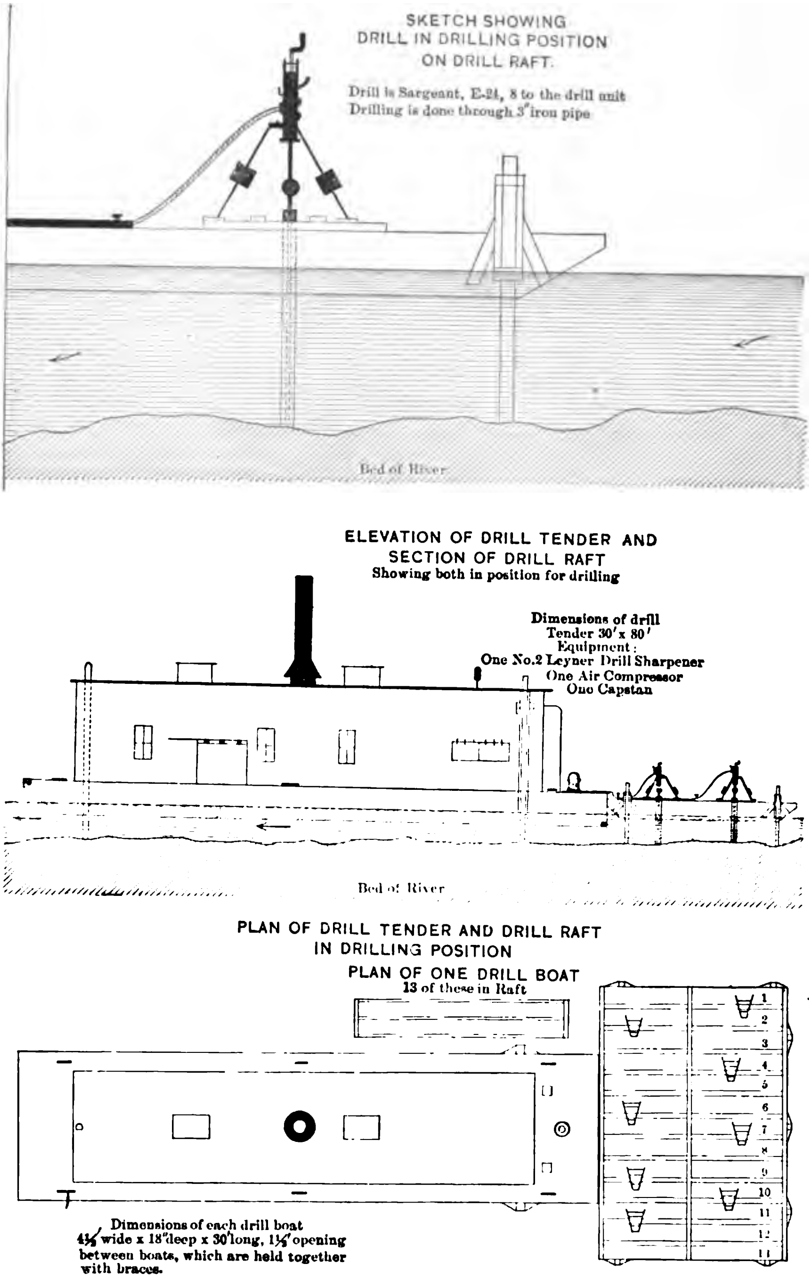


FIG. 229.—DRILL TENDER AND DRILLING SCOW, TENNESSEE RIVER.

essary to fill their places with new men. The November work was affected by two holidays, November 2d and Thanksgiving, and December by high water, which finally terminated the work for the season.

"The following method was universally followed during both seasons' work: The drill floats were placed in proper position for work by accurately lining them with ranges, marking the center of each half of the channel, and by cross ranges which marked the lower extremity of the drilling and blasting already done. A float thus lined is in position for drilling one-half the width of the channel for a distance of 25 feet, or length of the boats. At each setting, twelve lines of holes were drilled with six holes in each line. Each float carried eight drills, all of which are operated by steam.

"To prevent the holes from filling with gravel and silt, etc., the drilling was done through tubing or pipe, 3 to 4 inches in diameter. From three to five drill bits were used in putting down each hole, the first bit being 2½ inches in diameter and the last 1¾ inches. When the hole was down to a proper depth, a pipe that would exactly fit the top section of the hole was put in and the bit taken out and the hole loaded, the charge consisting of 80 per cent gelatin dynamite, and varying from 6 to 10 sticks according to the depth in the rock. The stick having the primer was placed about one-third of the way down from the top, having from 2 to 3 sticks on top of it and 4 to 6 under it. The charge was firmly packed down in the bottom of the hole with wooden poles which fit very closely the section of the hole. When the loading was completed the hole was marked by a cane, which was firmly embedded in the charge, leaving the top of the cane about 6 inches above the surface of the water and the primer wire looped around the top of the cane. When all the holes were loaded the primers were all carefully connected so as to make 3 circuits, 24 holes to each circuit, leaving the end wires of the first and fourth line looped around the top of the cane so that they might be readily found and connected with the lead. The float was then dropped down from over the holes and a set of lead wires attached to each circuit. When these were connected (insulated tape being used for making all these connections) the float and tender were dropped back about 250 feet below and the charges ignited by using three large batteries simultaneously.

"While each of the batteries would fire all these charges on shore, 24 holes was about their limit under water. This method was usually successful in getting off all charges together when the river was at a stage below 3 feet, but above this stage the connections were often interfered with by the force of the current and running drift, etc., necessitating several reconnections with the lead wire in order to get off the charges, and it was frequently the case that the wires were broken or withdrawn from the caps, making it impracticable to fire the charges.

"During the first season the drills were all sharpened by hand, but in 1912 a Leyner drill sharpener was installed which was operated with compressed air. This machine proved perfectly satisfactory, as it gave the bits more uniform gage, so that there was no difficulty of one drill following another, and the bit sharpened by this machine seemed to stand the hard service better than those sharpened by hand. A considerable saving was also effected, as one blacksmith and helper were able to sharpen steel for 24 drills, while three blacksmiths and helpers were necessary to do this work by hand.

"The seeming excessive cost of drilling here is due to the character of the rock, which is the very hardest of flint. It was found a very difficult matter to get steel that would stand this rock, and it was often necessary to change the

bits several times in getting down one sweep of the drill, which is only 24 inches. The slightest mistake in tempering would cause the bits to fail at once. In this connection it is interesting to note the difference between the rock at this place and the rock at Buck Island Shoals, 3 miles below this place, where rock excavation was carried on at the same time. The rock at Buck Island shoals is soft oolitic limestone which can be drilled easily and rapidly, and the drill bits keep sharp indefinitely. In addition to the ease with which it can be drilled, it is also very easy to break up. While it was found necessary to space the holes at Tuscumbia Bar 4 feet by 6 feet, a spacing of 7 feet by 8 feet at Buck Island was found ample, this spacing breaking the rock up better and leaving fewer large pieces than at Tuscumbia Bar. The same plant and methods were used at both places. The cost per linear foot at Buck Island Shoals was found to be $0.25 against $0.483 at Tuscumbia Bar.

"During the drilling season of 1912, 85,708 linear feet of holes were drilled and blasted. It is estimated that each linear foot of drilling loosened up .88 of a cubic yard and that 76,185 cubic yards were made ready for removal by the dredges at a cost of $0.543 per cubic yard for the drilling and blasting, and $\frac{3}{4}$ of a pound of dynamite was used for each cubic yard blasted.

"When the river was at a favorable stage for drilling it was found that an entire day with a double crew was required to drill out, load and detonate all the charges for one setting of the drill unit. Whenever unfavorable conditions occurred from weather or high water the progress was considerably lessened.

"The season's work for 1912 covered the heaviest part of the blasting, as it begun at the upper end of the extremely shallow water and extended entirely below it. While the number of linear feet of holes drilled in 1912 was about three times the amount drilled in 1911, it only covered about the same channel area, viz., 2500 linear feet. This was on account of the narrow spacing of the holes, the additional depth drilled and also that this season's drilling covered the heaviest part of the work. There is now left above the dam and area undrilled about equivalent to 1200 linear feet of channel.

	Linear Feet.
Average day's work for one drill unit operated with a double crew..	432
Average hour's work..	27
Average hour for one drill..................................	$3\frac{3}{10}$

"The drill unit used for work here consists of a float for carrying the drills, and a boat of some type having a boiler with sufficient power for furnishing steam for all of the drills. The floats have been modified to some extent, it being thought best to deck them, since with the decks they are less liable to sink during the heavy wind storms which we sometimes have.

"During the first season's work any boat having sufficient boiler power that could be spared from the plant was used as tender. The floats could be very quickly built and they were put to work in this way pending the building of a suitable tender. The type of boat built for this purpose is a barge 30 by 80 by 4 feet in depth, provided with three spuds. It is equipped with a 90-horse-power boiler, one Leyner drill sharpener, one compressor for operating same, and a steam capstan for handling the barge. The drill sharpener is Leyner No. 2, and the sizes of steel for forming the bits are $1\frac{1}{4}$, $1\frac{3}{8}$, and $1\frac{1}{2}$ inches. The sharpener is driven by air, the compressor in use being one manufactured by Chicago Pneumatic Tool Co., having a cylinder 9 inches for steam and air, by 11-inch piston

stroke, piston displacement 130 cu. ft. air per minute at 100 pounds pressure. This tender was built here and equipped for the work, the cost being as follows:

Building hull, cost of labor and subsistence	$1,346.66
Lumber and iron, nails, spikes, oakum, etc	1,902.40
One 90-horse-power Brownell boiler	900.00
One Leyner drill sharpener, No. 2	697.10
One compressor for running same	533.00
One receiver for air storage	64.80
One steam capstan	495.00
Setting up and connecting above	185.40
Building one-story cabin for sheltering machinery, material, and labor	465.00
Total cost of tender ready for use	$6,589.36
Type of float now in use, composed of 13 boats; cost, labor, and material	$1,140.00
Eight drills, E-24 Ingersoll-Sergeant	1,950.00
Steam hose, drill steel, iron pipe	365.00
Cost of float, equipped for drilling	3,455.00
Cost of tender	6,589.06
Cost of one drill unit	$10,044.36

"Three drill units were operated during the season of 1912, and it was found that one drill sharpener could keep steel in shape for the three units, each of which carried eight drills. Only one tender, as described above, has been built for the work here, the other two units being furnished with steam by spare pieces from the plant. It was found that the greatest wear and deterioration in the drill units are in float and the drills, it having been found necessary to rebore and overhaul a number of the latter and that two seasons' work is about all that the floats will stand.

"The area drilled in 1911 is shown on accompanying sketch in solid black, extending from the upper end 2500 feet down-stream. Estimated in place there are 35,136 cubic yards of material to be removed in order to reduce this area to grade. As previously stated, when this area was dredged a great many high points were found, making it necessary to reblast a considerable portion of this channel, and suggesting the advisability of deeper and closer drilling. During this season 28,831 linear feet of holes were put down, loosening up about 23,155 cubic yards of material, costing as follows:

Actual field cost, including material, salaries, and subsistence, etc.	$13,299.59
Deterioration of plant on account of season's work	3,840.00
Overhead charges	664.98
Total cost	$17,804.57
Cost per linear foot	0.617
Cost per cubic yard, loosened	0.76

"The hatched area on sketch shows portion of channel drilled in 1912, on which the dredges are now working. The closer spacing of the holes, 4 by 6

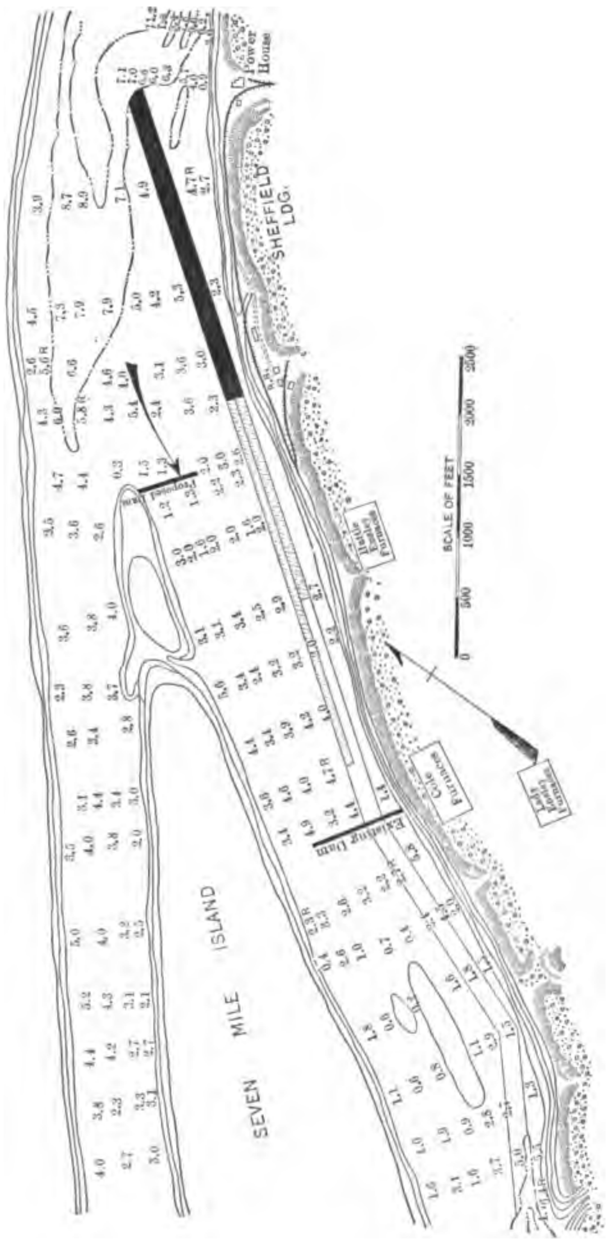


FIG. 230.—TUSCUMBIA BAR, TENNESSEE RIVER. BLACK DRILLED IN 1911. HATCHED DRILLED IN 1912.

feet, has served to loosen up the material much better and the dredges are able to get 'grade,' with the exception of a small area which was drilled when the river was too high. During the season of 1912 85,708 linear feet of holes were drilled and blasted, loosening up 76,185 cubic yards of material estimated in place, costing as follows:

Entire field cost, material, etc.	$41,457.75
Estimated deterioration of plant	7,500.00
Overhead charges	2,072.88
Total cost	$51,030.63
Cost per linear foot, drilled and blasted	.595
Cost per cubic yard, loosened	.677
Total amount of material loosened up in the two seasons 99,566 cubic yards.	
Total amount of dynamite used, 75,450 pounds	$12,855.36
Amount of dynamite per cubic yard, ? pound	.127

Should a regular drill scow be available, such as is in use on the harbor work of Victoria, B. C., (Fig. 231) it will be unnecessary to rig up scows and drills as outlined in the preceding pages. The Department of Public Works of British Columbia operates this drilling plant, and during the fiscal year 1911-12, 1690 cubic yards of rock were removed at a cost of $6.13 per cubic yard. The entire dredging plant is under the superintendence of H. A. Bayfield, M. Can. Soc. C.E. This department is also using a Lobnitz rock breaker which was operated only part of the year, removing 1000 yards of rock at an approximate high cost of $9 per cubic yard. This plant, however, was not operated enough to give it a fair test, and considerable trouble was had in mooring it with cables, on account of having to slack the cables to allow the passage of vessels, and spuds have been added to facilitate the work. The largest cutter, weighing 20 tons, is for use in depths up to 40 feet. Considerable trouble was also had with the cable breaking, and this is doubtless due to the causes mentioned hereafter, and can, doubtless, be overcome as explained by the makers of these machines.

The winch should be stopped before cutter is let go, and should be started slowly to take the weight on the hoisting rope without sudden jerk. When fitting a new rope on the winch, before starting regular work, it is necessary to take the twist out of the rope as follows:

The cutter should not be raised and dropped, otherwise the rope will kink and tie itself into knots, which destroys the rope.

On the contrary, the weight of the cutter should be taken on the rope by raising the cutter with the winch slightly, say the lower

end of the cutter about 1 foot from the rock. The cutter will then begin to turn. It should then be lowered on to the bottom, so as to stop the speed of spinning. And when it stops turning it should be raised again about 1 foot.

This should be done several times until all tendency of spinning has gone.

The cutter can then be raised 2 or 3 feet, and dropped, care being taken to catch the rope immediately the blow is struck, to prevent the rope running out.

FIG. 231.—VICTORIA DRILL SCOW AND TENDER.

By this procedure it will be found that the man operating the winch will be able to entirely avoid kinking the rope after a little practice and experience.

The method of breaking rock by impact has been employed for over twenty years in similar work, and machines of this kind are manufactured by Lobnitz & Co., of Renfrew, Scotland, which has a cylindrical ram with a projectile-shaped cutter-head, while the Krupps of Germany make one with a square cutter-head that sharpens like a cold chisel. These Lobnitz rams, shown in Fig. 232, weigh from 6 to 15 tons, and are raised from 6 to 15 feet and dropped about four times a minute. As the whole force of the

blow is delivered on a very small surface, the tempered points of the cutter crush the very hardest rock. From 10 to 20 blows are required at each point and then a move of 3 feet is made. The rock is partly broken and partly pulverized, and after one layer of about 3 feet in thickness is broken up, the bottom is cleaned up with a dredge before another layer is started upon, unless the area is large enough for the breaker to work in one place, while the dredge is cleaning up in another. The machine shown in Fig. 233 has the cutter at one end of the scow, while the one shown in Fig. 234 operates through the center of the scow. The points shown in Fig. 235 are

FIG. 232.—LOBNITZ ROCKCUTTER RAM.

both a new point, and one pretty well worn down, showing that it is self-sharpening, on account of the center being harder than the outside.

The records show that in many places where hard rock has been removed, that 2 cubic feet of rock are broken per blow, to a size that can be readily dredged. The average number of blows per hour is usually about 150, and will give about 10 cubic yards per hour for a single-cutter machine.

The cost of operating one of these tools can be arrived at from the fact that four men are required to run it, one ton of coal per

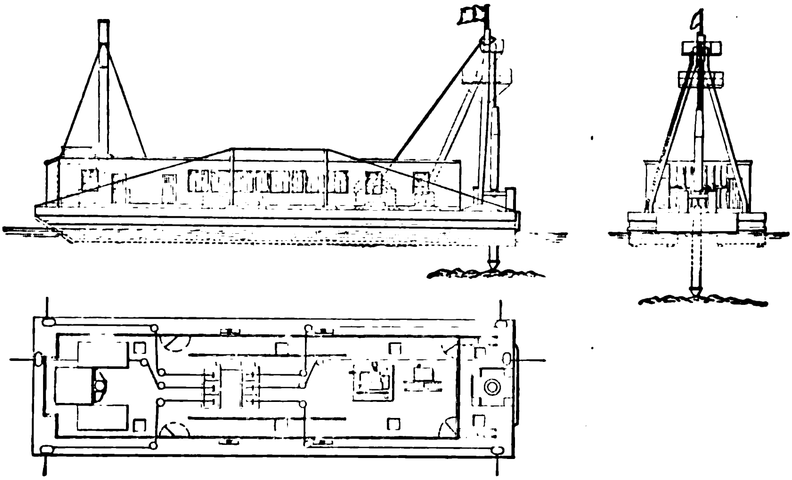


FIG. 233.—LOBNITZ ROCKCUTTER WITH TIMBER HULL.

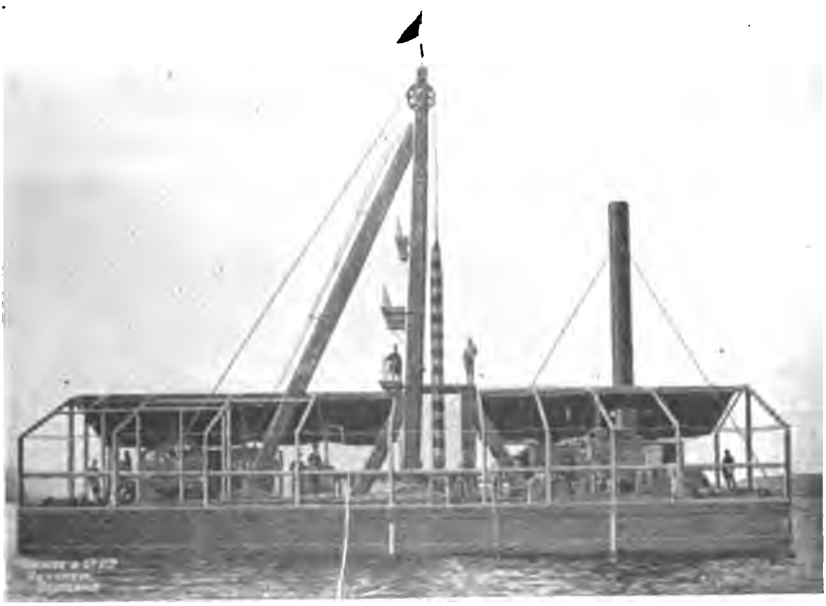


FIG. 234.—LOBNITZ ROCKCUTTER WITH CENTER WELL.

FIG. 235.—LOBNITZ ROCKCUTTER POINTS, BOTH OLD AND NEW.

FIG. 236.—ROCK BROKEN BY 15-TON ROCKCUTTER, DREDGED AND DUMPED ASHORE

day is consumed, and an additional amount equal to three times the cost of the coal is usually expended on oil, supplies, and repairs, although this latter item may run considerably in excess of this. The actual cost of removing sandstone rock at practically the rate given above, for Blyth Harbor Works in England was at the very low cost of 31 cents per cubic yard. French records show that hard granite has been removed at the rate of 3 yards per hour by a similar machine, indicating that the cost would run to about $1 per cubic yard.

Such machines are more particularly suited to removing shelly rock, or leveling off the site of a pier, than any other type, if one happens to be available; but in most cases they will be much more easily handled if provided with spuds instead of mooring lines, in accordance with the experience before mentioned in describing the one in use at British Columbia.

A pile of rock as broken up by one of these breakers is shown in Fig. 236 after it was dredged and put ashore.

CHAPTER XVII

DIPPER AND LADDER DREDGES

THE dipper dredge, next to the clam-shell, is the most likely one that an engineer would employ in digging out for the foundations of engineering structures, especially if the material is so compact or hard that the clam-shell will not handle it economically. For soft material the bucket, which is similar to the ordinary steam-shovel bucket, can be fitted with a rounded plate nose or lip, but for medium and hard digging, it must be fitted with teeth. The small dipper dredges of the Osgood type, Figs. 237 and 238, are among the best machines of this kind in use. Such dredges are more simple in construction than elevator dredges described in the latter part of this chapter, and are consequently easier and cheaper to keep in repair. The hull of this dredge is $70 \times 17 \times 6$ feet deep, with two 6-foot pontoons which can be removed to allow the dredge to go through a lock. The engines consist of a double-drum main engine with 8×10 cylinders; a swinging engine with 6×8 cylinders; and a crowding engine with 5×6 cylinders, which are all used in operating the $1\frac{1}{4}$-yard bucket on the steel boom 45 feet in length.

The crowding engine is used to control the dipper and enable it to make a practically level bottom at one cut, and also thrust the dipper far enough beyond the boom to allow it to dump 52 feet from the center. This dredge, which cost complete only $10,000, is operated by a crew of only four men, and consumes but 1 ton of coal per day of twelve hours. The average excavation during four months' work was 549 cubic yards per day, or a probable cost of 12 cents per cubic yard without tugs or scows. The machine has sufficient power to dig hardpan, boulders, and very soft shale rock. For the small amount of dredging that would likely have to be done in foundation work, the cost would probably run, for comparatively easy digging, from about 25 to 40 cents per cubic yard. These latter figures cover the use of dump scows, and a tugboat for towing them, but the low cost given above is for simply dumping the material as far as the boom will reach. This can sometimes be done

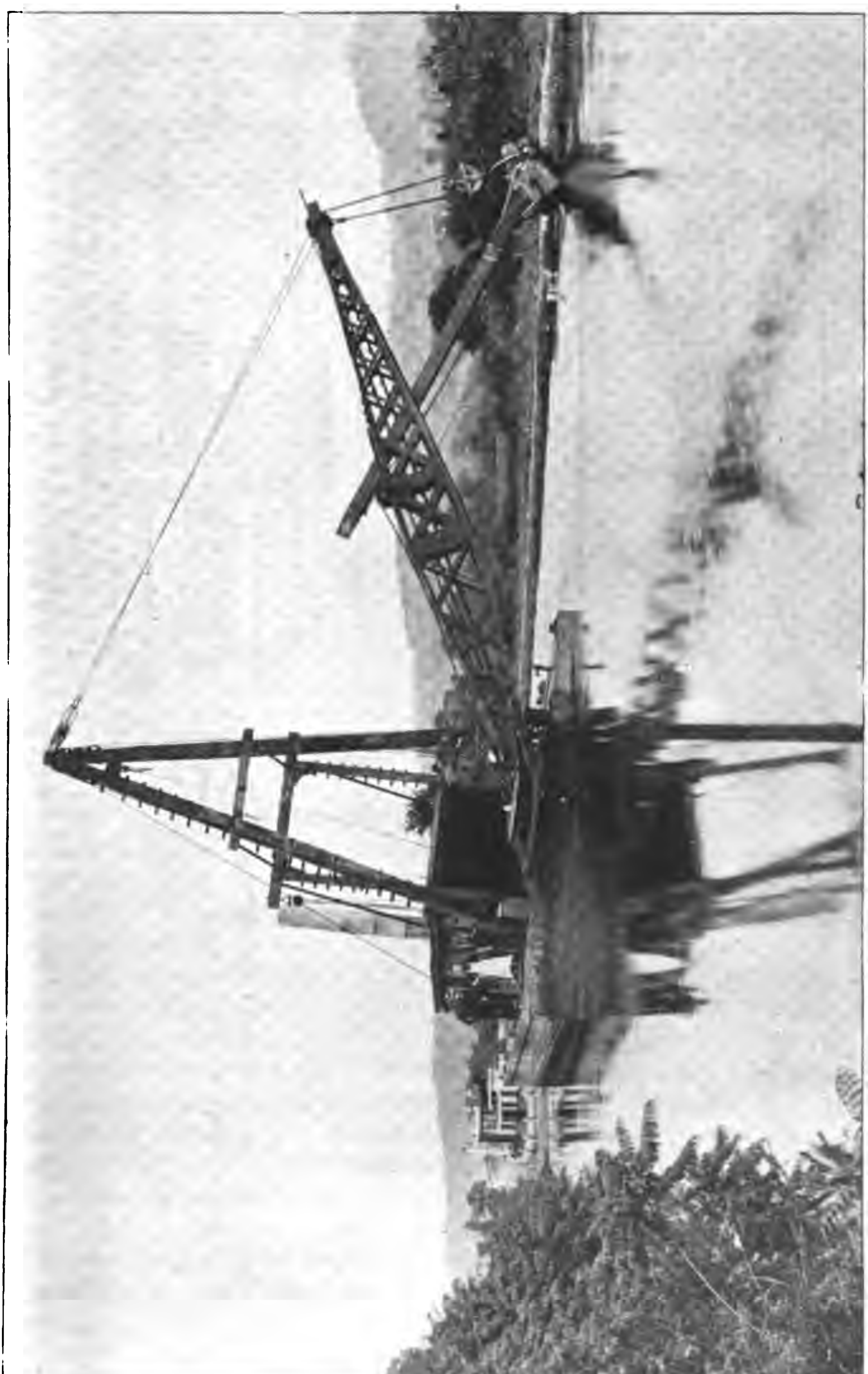
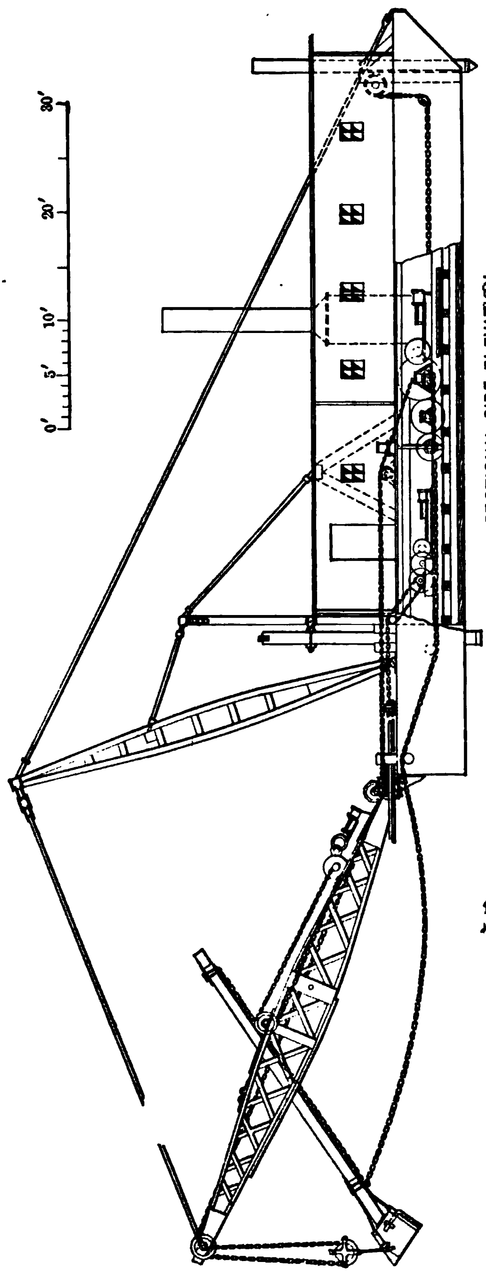


FIG. 237.—OSGOOD DIPPER DREDGE, NEW YORK STATE CANALS.



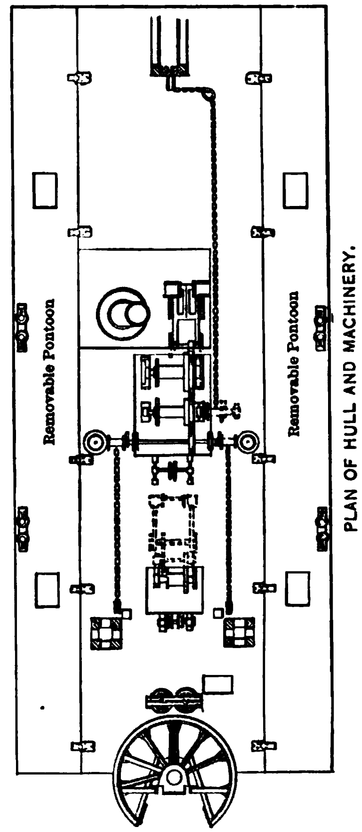


Fig. 238.—Osgood Dipper Dredge, Details, New York State Canals.

where the current will carry away the dredged material; where the water is deeper than required for navigation; or wherever the contractor has permission from the Government engineers to dump. The data of small-sized dipper-dredges is shown in Table XXXIII.

TABLE XXXIII.—SMALL-SIZED MARION DIPPER DREDGES

Bucket Cap. in Cu. Yds.	2½–3½	2½–3½	2½–3½	1¾	1¾
Cu. yd. cap. in 10 hrs.	600–1400	700–1600	800–1800	600–1200	700–1400
Length of boom......	45 ft.	40 ft.	35 ft.	40 ft.	35 ft.
Will dig below water level..............	25 ft.	22 ft.	20 ft.	22 ft.	20 ft.
Will dump above water level..............	12–15 ft.	10–12 ft.	8–10 ft.	10–13 ft.	8–11 ft.
Cen. of boat to cen. of dump.............	40–45 ft.	35–40 ft.	30–35 ft.	35–40 ft.	30–35 ft.
Length of hull........	80–90 ft.	75–80 ft.	70–75 ft.	65–70 ft.	75–70 ft.
Width of hull........	30–35 ft.	26–30 ft.	23–26 ft.	22–25 ft.	21–24 ft.
Depth of hull........	7–7½ ft.	7 ft.	7 ft.	6½ ft.	6½ ft.
Size of engines.......	two 12×14	two 12×14	two 12×14	two 10×12	two 10×12
Hoisting cable........	1¼ in.	1¼ in.	1¼ in.	1⅛ in.	1⅛ in.
Swinging cable.......	1⅛ in.	1⅛ in.	1⅛ in.	1 in.	1 in.
Backing chain........	1 in.	1 in.	1 in.	⅞ in.	⅞ in.
Size number of boiler..	No. 13	No. 13	No. 13	No. 12	No. 12

Boiler No. 13. Locomotive Type.—54 in. diameter; firebox, 64 in. long, 52 in. high, 49 in. wide; 60 3-in. flues, 150 in. long.
Boiler Ne. 12. Locomotive Type.—48 in. diameter; firebox, 64 in. long, 52 in. high, 43 in. wide; 56 3-in. flues, 144 in. long.

The small dredge just described is shown with chains to operate it, but the larger and later dredges practically all have a single-wire rope running to the dipper, or, in case the engines are not powerful enough for this, a single sheave can be used to carry a double line from the end of the boom to the dipper.

The dredge shown in Fig. 239 is one constructed by the author for use on Puget Sound. It has a hull 40×100×11 feet 6 inches deep. The sides are 12×12 in thickness; the bottom planking is 5 inches thick, and the deck planking 4 inches thick. The keelsons are 12×14, and the deck beams, running longitudinally, 10×12, supported every 3 feet 6 inches. The longitudinal bracing consists of two intermediate solid bulkheads of 12×12 timbers, and two Howe trusses running the entire length of the hull, and placed 20 inches in the clear inside of the sides of the hull. Heavy knee bracing of natural fir knees was used at every other beam longitudinally, at every beam across the end, and at every other beam across the stern. The hull was also braced transversely with a Howe

Fig. 239.—Dipper Dredge on Puget Sound.

truss 8 feet inside of the front end. All of the fastenings of the hull, and the calking and painting was of the very best throughout. The boiler was of the locomotive type, built under marine inspection, and having 2100 square feet of heating surface. The machinery was built of a size and strength sufficient to carry a 7-yard dipper for easy digging, but was really equipped with a 3½-yard dipper for hard digging, and a 4-yard dipper for medium digging. Everything

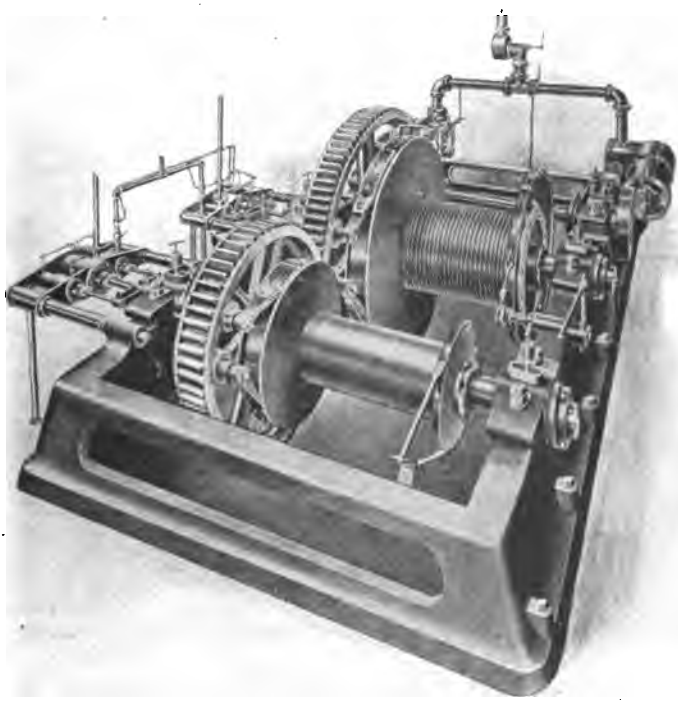


FIG. 240.—MAIN DIPPER DREDGE ENGINES. 16×20.

was operated by levers from the pilot house. The cost of the hull was approximately $18,000, ready to receive the machinery and the house, while the dredge complete, ready for operation, cost $56,000. Three 300-cubic yard dump-scows were constructed, to carry away the material, at a cost of approximately $6000 each; and a 75-foot tugboat was used to tow the scows from the dredge to the dumping ground. The machinery on the dredge, of the Featherstone type, consisted of main dredge engines, double-cylinder, double-

drum, as shown in Fig. 240. The reversible swinging engines were double-cylinder engines as shown in Fig. 241. The spud engines are of double reversible type, and hoist the spuds with wire ropes, and pin up the dredge or hold it up on the spuds solidly for digging, by means of wire ropes running over sheaves at the top of the spuds so as to lift the hull on the spuds, as much above the natural flotation line as is necessary to give the dredge the necessary stiffness while digging. The rear spud engines were used of a design to lift the spud, and separate cylinders to walk the dredge ahead. The other equipment for the dredge consisted of surface condenser, air-pump and hot well, circulating pump, boiler-feed pump, and a fire-pump. The dredge was lighted by an electric-light plant large enough to operate

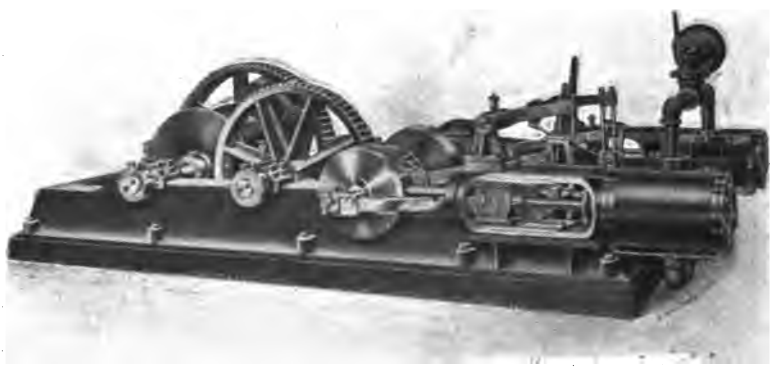


FIG. 241.—SWINGING ENGINES FOR DIPPER DREDGE. 10×14.

sixty 16-c.p. lamps, and at the same time furnish current to a 3000-c.p. search-light. The specifications for the dipper are as follows: "The dipper will be made of ½-inch steel plate; the mouthpiece being of forged steel 2 inches thick. The dipper (Fig. 242) will be provided with four dipper teeth with tool-steel points (Fig. 243); they will be let into the mouthpiece of the dipper and fastened to the shell with wrought-iron staples, so that they can be easily removed for sharpening. Each tooth will weigh approximately 1000 pounds.

"The dipper will be trimmed in the most substantial manner with heavy cast-steel trimmings, and all necessary lugs to support pins. The bail hinge, arm and brace pins will be of extra size; all bushed with steel, and pressed tightly into position. The dipper arms, braces and bail will be made of best hammered iron. The door

will be made of two steel plates, each $\frac{5}{8}$ inch thick, and will be provided with a relief door and all necessary latches, latch rollers, roller cages, catches, etc., for convenience in handling and quick operation. The arms are provided with six bolts for fastening to the dipper handle. The handle trimmings of the dipper will consist of the following: Staple, washers and nuts for backing chain, trip levers, and necessary attachments for same. For the lower end of dipper handle provide the latest form and design and reinforcements for dipper arms. On each side of the handle will be provided two

FIG. 242.—EIGHT-YARD DIPPER WITH LIP.

$22 \times 1 \times 84$-inch plates, to which will be fastened four $5 \times 3\frac{1}{2} \times \frac{1}{2}$-inch angles placed just wide enough apart to hold the dipper arms."

The record of the dredge in medium hard digging is about 30,000 yards per month, where the material had to be towed a distance of $1\frac{1}{4}$ miles. The comparative amounts dug by dipper dredges is shown in Table XXXIV.

The greatest dredges of this type have undoubtedly been constructed for use on the Great Lakes, in the central portion of the United States. The dredge *Toledo*, constructed in the last

few years, has a dipper of 15 yards capacity. This dipper is operated with a single wire rope 3 inches in diameter, as shown in Fig. 244, which also shows the longitudinal steel stiffening trusses which are commonly used in wooden hulls, this hull being 44×135×13 feet 6 inches average depth. The machinery is all placed in the hold, with the exception of two small winch engines for handling the scows, which are placed on deck. The main engines are horizontal, reversing, twin tandem compound condensing. The high-pressure cylinders have piston valves, while the low-pressure have flat slide valves. The

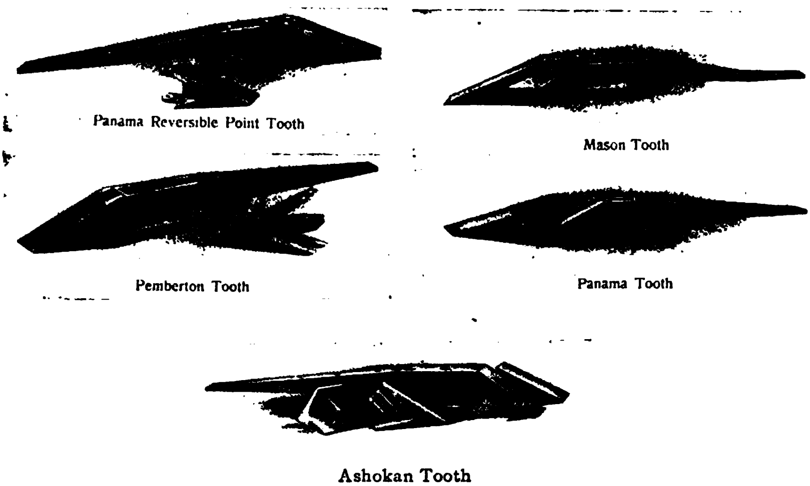
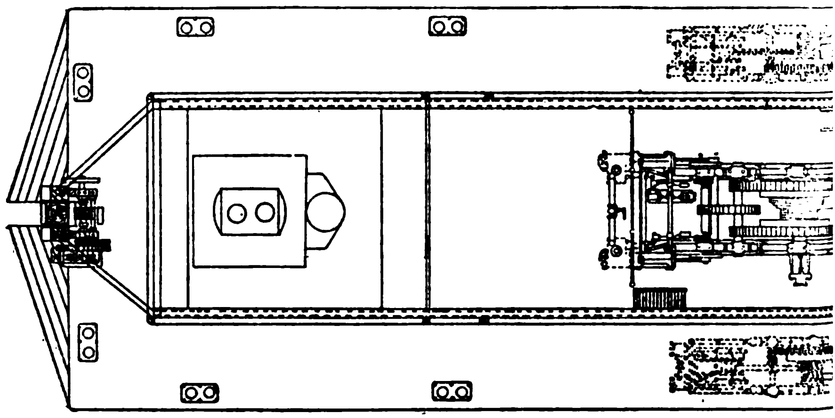


FIG. 243.—DIPPER DREDGE TEETH.

enormous capacity of the dredge makes it necessary to provide scows of 1500 yards capacity to take away the material. The average running time of dipper dredges is from sixteen to eighteen hours out of twenty-four.

Dredges of the ladder or elevator type (Fig. 245, being a dredge constructed by Lobnitz & Co., for the Suez Canal), are very seldom used in the United States, where dredges of the dipper and suction type have reached such a high state of development and efficiency. One of this kind, operated by the Department of Public Works of British Columbia, removed during the fiscal year 1911–12, at Vancouver, B. C., 621,310 cubic yards at a cost of 16 cents per cubic yard. This cost is made up as follows:



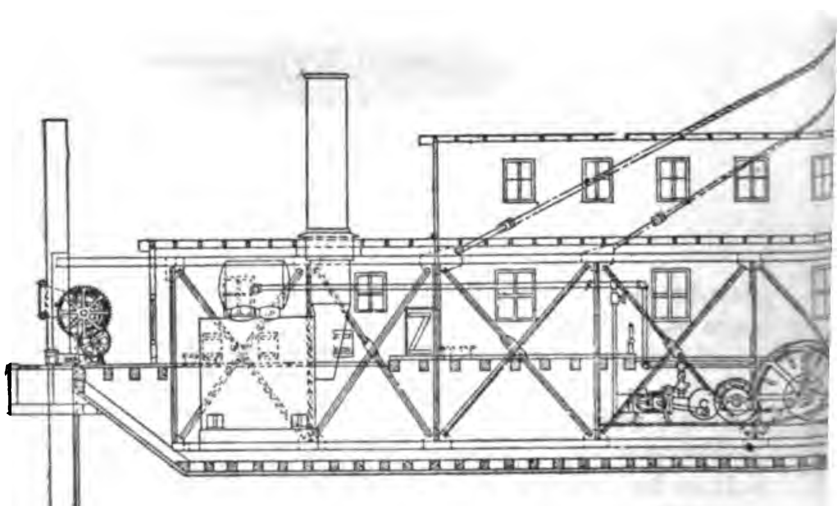
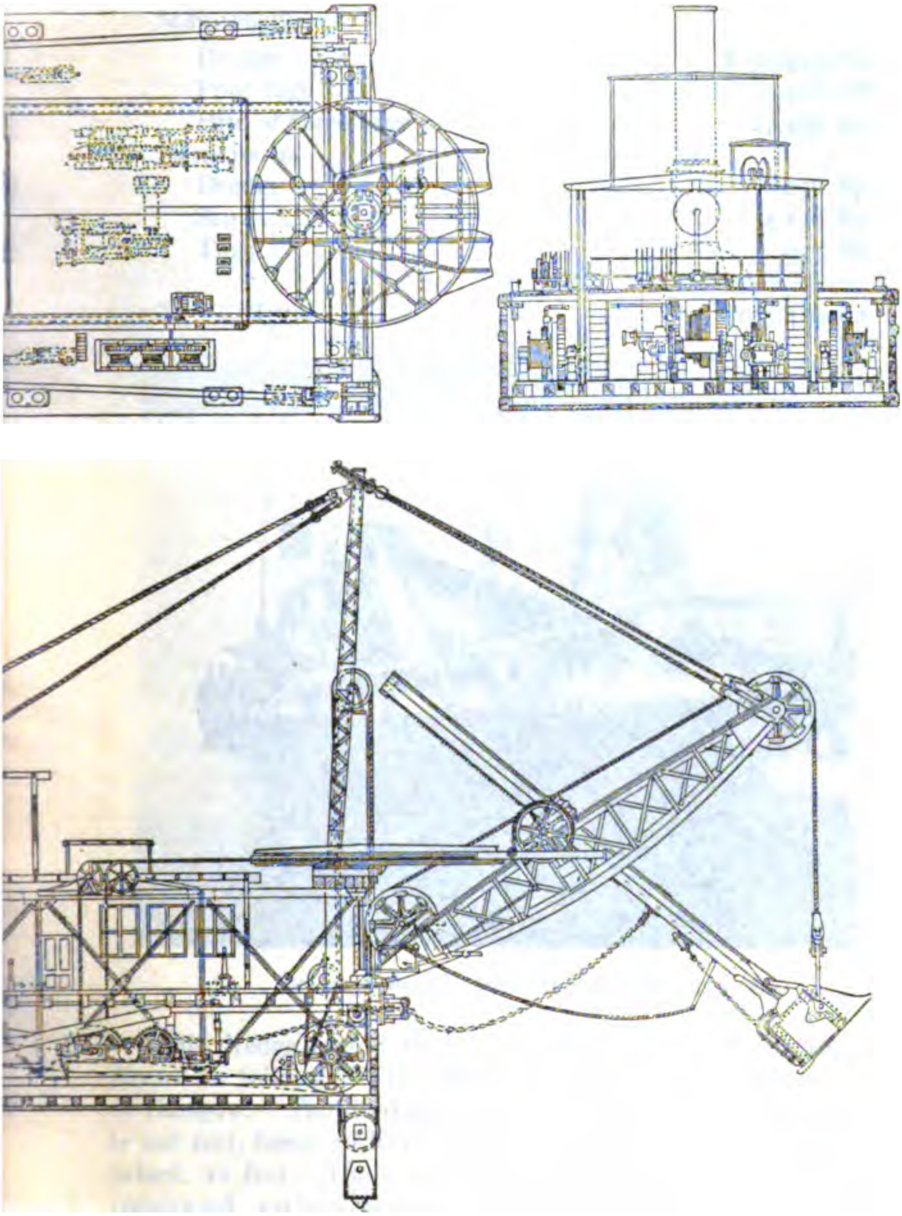


FIG. 244.—DIPPER DRED



'OLEDO," WITH 15-YARD DIPPER.

(*To face page* 33c.)

Maintenance of	
Dredge	$ 53,445.65
Four tugs	9,412.16
Hire of three tugs	27,450.91
Repairs to	
Dredge	8,036.67
Scows	4,136.87
Tugs	937.66
Total	$103,419.92

FIG. 245.—BUCKET DREDGER FOR SUEZ CANAL CO.

This dredge is described in a recent number of the *Engineering Record* as follows: "The *Mastodon* was built by Simonds & Sons, of Glasgow. The dredge is of steel throughout; the length overall is 206 feet, beam 36 feet 6 inches; maximum draft, with the ladder raised, 12 feet. The main engine equipment consists of a pair of compound surface-condensing engines for operating the bucket line or for propelling the ship. These engines are so arranged that either one can be used to drive the elevating machinery. In order to balance wear and facilitate overhaul, the bucket line is usually driven by one engine for a week, and then the second engine is used

for a similar period. As the engines each develop about 600 indicated horse-power, the operation of the elevator gives only a comparatively light loading. Indicator cards taken while the elevator brought up fifteen loads per minute from a depth of 40 feet showed an average of 206 indicated horse-power.

"There are forty-eight buckets in the elevator. These have a capacity of 24 cubic feet each and are equipped with manganese-steel replaceable lips. The ladder is long enough to permit operating in depths up to 50 feet, and is said to work satisfactorily in heavy gravel, carrying boulders as large as the buckets will bring up. The *Mastodon* was purchased primarily for widening the First Narrows at Vancouver, but has been assigned to various other stations where it was required to dredge heavy gravel and small boulders. The large amount of time lost waiting for scows is chiefly due to the fact that in the First Narrows the tidal currents range as high as 9 knots per hour, which interfered with the speedy handling of scows. The repairs to dredge includes alterations so extensive as to amount practically to remodeling, deemed necessary to better adapt the vessel for the work in hand. These alterations were subsequent to the loss of 54.9 per cent. of working time for the first month that the *Mastodon* was in commission, which time was spent adjusting machinery. Since the tabular data were compiled the *Mastodon* has been converted into an oil burner, and the time lost on account of coaling has become *nil.* Two months were spent working in exceptionally hard material, some of which was hardpan and rock, which could only be scraped over, and during this time the yardage per hour of actual dredging fell to approximately 280 cubic yards. Due to this fact the average capacity is somewhat lower than could normally be expected."

The elevator dredge *Viscount Ridley* is of the hopper type, and is described in a recent number of the *Engineer:* "The deepening of the harbor and entrance channel at Blyth, so as to have 24 feet at low water or 39 feet at high water of spring tides, involves the removal of about 500,000 cubic yards of rock broken with a Lobnitz breaker. For dredging this broken rock the Blyth Harbor Commissioners have recently introduced the powerful dredger *Viscount Ridley*, which has been built by Fleming & Ferguson, Ltd., of the Phoenix Works, Paisley. This vessel (Fig. 246) has a length of 136 feet, a breadth of 30 feet, and a depth molded of 11 feet 6 inches. She is specially constructed with heavy scantlings so as to withstand the abnormal strains of rock dredging, and is capable of lifting rock at depths varying down to 45 feet below water-line. The engines

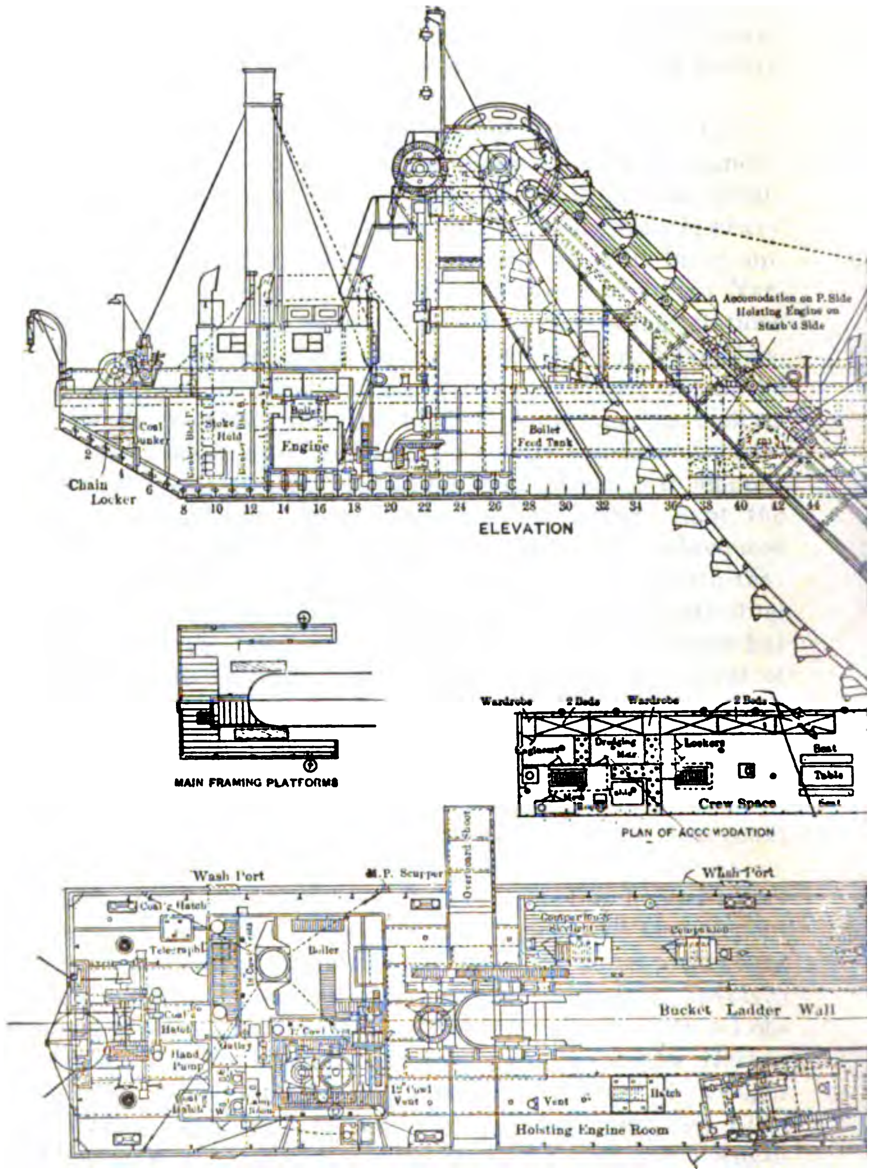
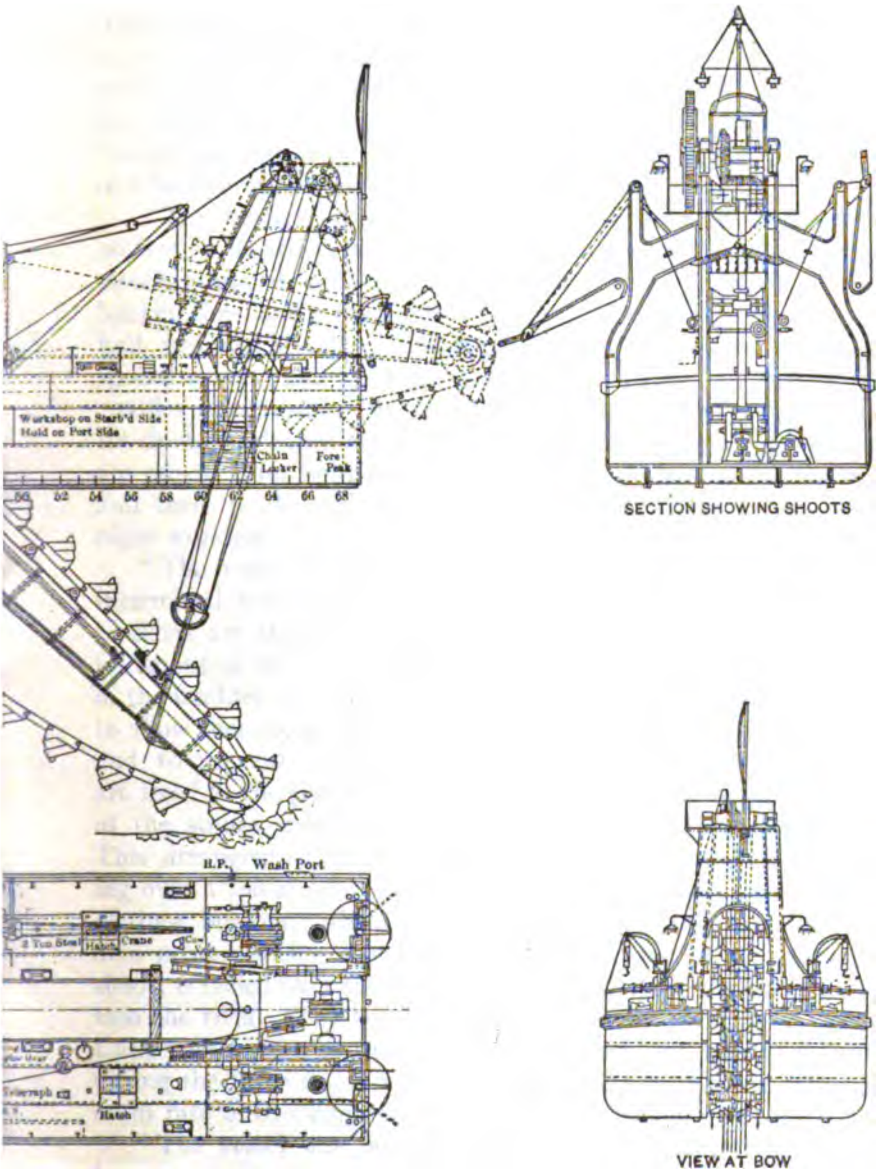


FIG. 246.—THE ELEVATOR OR LADDI



HOPPER DREDGE "VISCOUNT RIDLEY."

(To face page 338.)

for driving the main gear are coupled direct to the gear shaft, and are supplied with steam from one large cylindrical return-tube boiler. The shaft for driving the bucket chain is driven through cast-steel spur and bevel gear, the teeth all being machine cut. These, together with the shaft, brackets and framing, have a large margin of strength over the power developed by the engines, so that in the event of the buckets getting such a hold on the rock as to stop the gear, the risks of a breakdown are largely reduced.

"The bucket ladder is constructed of mild H-section girders, 96 feet in length between centers and 6 feet deep at center, and is strongly braced, while the chain of buckets is of special design, the bucket rims being reinforced by hard-steel cutting lips. Both the hull and machinery of the vessel were constructed under Lloyds' special survey, and the boat is fitted with a complete installation of auxiliary machinery. To maneuver the vessel, three specially powerful dredger winches are fitted on deck. Independent steam engines are provided for hoisting and lowering the bucket ladder and shoots, and there is also an electric installation for lighting the vessel for night working.

"The rock in Blyth Harbor consists of very hard sandstone intermixed with veins of metamorphic rock, and large numbers of boulders are also met with, a large number of these boulders being taken out of the buckets in the space of three hours. As the removal of the boulders from the buckets entails a loss of time, it was decided to allow them to go down the shoots into the hopper barge alongside, and, to prevent them from damaging the barges, strong channels are fixed to the points of the shoots in order to minimize the impact of the stones striking the hopper beams and chains of the barge. This arrangement had proved very successful, and stones weighing over a ton regularly pass down the shoots into the barges. One of these boulders, which, owing to its shape, came down the shoots at a great velocity, and, striking the guard bar at the point of the shoot, traveled over the hopper on to the deck of the barge and then into the river. The guarantee undertaken by Fleming & Ferguson, Ltd., was to raise 200 tons of rock per hour, and this was exceeded during the three days' consecutive trials just completed. The maximum rate of dredging obtained was at the rate of 273 tons per hour.

"The vessel was built to the instructions and requirements of Messrs. J. Watt Sandeman & Son, Newcastle-upon-Tyne, the engineers for the Blyth Harbor Commissioners, and the trials were carried out under their supervision and that of Mr. G. D. McGlashan, the resident engineer at Blyth."

CHAPTER XVIII

SUCTION AND HOPPER DREDGES

The two preceding chapters have covered, as thoroughly as is necessary for the subject of foundations, the construction and operation of clam-shell, ladder, and dipper dredges, and if any cases should arise where channels are to be dredged, along the sides of which are to be built the foundations of sea-walls, and any locations dredged out for large structures such as drydocks and locks, it would seem well at this point to discuss, preferably, the kinds of work to which the various dredges are best adapted.

The clam-shell dredge is of the greatest use in the softer kinds of material, and where the débris has to be towed away for dumping, or else has to be deposited on the bank on shore with a long boom. This dredge is the only type where dredging can be done economically to depths greater than about 50 feet. Where the depth is greater than 50 feet, it will, however, require the very best levermen to get out enough yardage to keep the work on an economical basis.

The dipper dredge cannot be used for a greater depth than about 50 feet, but with powerful machinery and small dippers fitted up with teeth for digging stiff clay, hardpan, cemented gravel, or even soft rock, it is the only machine to be seriously considered, and where the depths are not too great, it will be fully as economical, if not more so, than the clam-shell. Of course the material must be towed away, except in rare cases where it can be deposited on the bank by means of a long dipper handle, as shown in Fig. 237. Where the digging is not too hard, the larger sizes of dippers can be used to advantage, and will greatly reduce the cost of the work.

The ladder dredge, being so much more expensive to operate and keep in repair than clam-shell or dipper machines, is seldom used in the United States, and work that cannot well be done by the clam-shell or dipper, is in the author's opinion much better carried on by a suction dredge.

The suction dredge is the only one that will economically handle material to considerable distances on shore, either for the wasting

of the material, or for reclaiming low lands or tide lands. The most economical size of this dredge is one having a 20-inch centrifugal pump, as shown by the table.

Such a machine will handle in twenty-four hours upward of 20,000 cubic yards of mud to a maximum distance of about 6000 feet, and will handle 8000 to 10,000 cubic yards per day of light sand through a pipe line less than 1000 feet long, and for any material running from mud up to moderately heavy sand and gravel, it is the only machine that should be used where large quantities of material are to be removed, and where there is a place to deposit the same ashore, or for filling in shallow water. The average running time is usually over twenty hours out of twenty-four.

The data given in Table XXXIV is made up simply as a matter of judgment by the author from experience gained through many years of construction and operation of all kinds of dredges on a large scale, and while the distances for towing the material and discharging it ashore are only approximate, the quantities given are very close averages for twenty-four-hour operation with two shifts. The cost of excavation will be discussed in a later chapter, from which can be gathered further information as to the best machine to use in any given location.

TABLE XXXIV.—CAPACITY OF DREDGES OF DIFFERENT TYPES

	Mud.	Mud and Sand.	Light Sand.	Heavy Sand.	Sand and Gravel.	Heavy Gravel.	Stiff Clay.	Hardpan.
Clamshell 2½ yd.	2 mi. 2000 yds.	2¼ mi. 1500 yds	2½ mi. 1200 yds.	3 mi. 600 yds.	3 mi. 600 yds.	4 mi. 350 yds.	6 mi. 250 yds.	
Dipper 3 yd.	2 mi. 3000 yds.	2½ mi. 2400 yds.	2½ mi. 2400 yds.	3 mi. 1800 yds.	4 mi. 1500 yds.	5 mi. 1200 yds.	5 mi. 1000 yds.	7 mi. 500 yds.
Dipper 6 yd.	3 mi. 5000 yds.	3 mi. 5000 yds.	3½ mi. 4500 yds.	4 mi. 3500 yds.	6 mi. 2500 yds.	8 mi. 2000 yds.	8 mi. 1800 yds.	
Ladder Dredge	....	....	....	5 mi. 2500 yds.	5 mi. 2500 yds.	6 mi. 2000 yds.	8 mi. 1500 yds.	
Suction 20 in.	6000 ft. 20000 yds.	5280 ft. 4000 yds.	5000 ft. 2000 yds.	3000 ft. 2500 yds.	2000 ft. 1000 yds.	800 ft. 600 yds.	1000 ft. 1000 yds.	
Suction 30 in.	6000 ft. 30000 yds.	5280 ft. 8000 yds.	5000 ft. 4000 yds.	3000 ft. 5500 yds.	2000 ft. 2200 yds.	1200 ft. 800 yds.	1200 ft. 2000 yds.	

Capacity in cubic yards per day, maximum tow in miles and length of pipe line in feet.
Clam-shell dredge, 2 250-yd. scows. 50 horse-power tug.
Dipper dredge, 3 yd., 3 300-yd. scows. 75 horse-power tug.
Dipper dredge, 6 yd., 3 500-yd. scows. 125 horse-power tug.
Ladder dredge, 3 500-yd. scows, 125 horse-power tug.

The suction dredge is essentially an American tool, and has reached a high state of development on almost entirely a practical basis, though theoretical considerations are receiving more attention than formerly. While it is stated above that a 20-inch machine is the best for all-around purposes, those as small as 12 inches are very often used as a makeshift, and in soft material get out as high as 100,000 cubic yards per month. With a larger size than 20 inches there is trouble in stiff digging to cut as much material as can be pumped away, so that the larger sizes can be said to be of greater service only in case the material is comparatively soft, or with a shorter pipe line where the material is not of the softest. The 20-inch machines constructed and used by the author were of the type shown

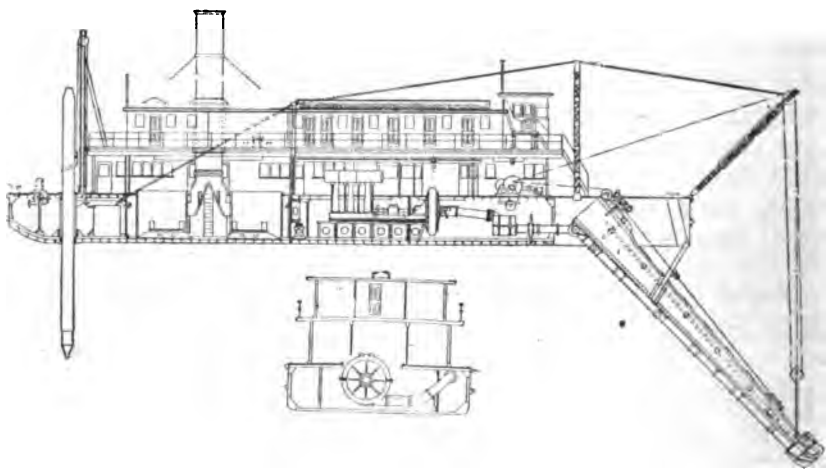


FIG. 247.—PANAMA CANAL SUCTION DREDGE.

in Figs. 247 and 248. The dredge, Fig. 247, was constructed by the Ellicott Machine Co. for work on the Panama Canal.

The machine is of the ordinary type with a round-nose cutter, and a single centrifugal pump which is directly connected to a triple-expansion engine similar to the compound one shown in Fig. 249 for the sea-going dredge *Galveston*, although the author usually prefers a compound engine with a ratio of between $2\frac{1}{4}$ and $2\frac{1}{2}$ between the high-pressure and the low-pressure cylinders for a 20-inch machine, as there will then be little difference in the economy of the two where the steam pressure is from 170 to 200 pounds per square inch, and the compound engine will have a less number of working parts to care for, and consequently a greater running time will be devel-

oped by the plant, on account of less stoppage for repairs, which is one of the biggest items to look after in a plant of this kind.

The machine is kept into the cut by wire ropes running out to piles or anchors on either side of the bow, or from the end of the ladder, operated by winding machinery placed on the upper deck forward as shown, and controlled from the pilot house, as is all the rest of the machinery and equipment. The machine swings about the main spud aft as a pivot, and is moved forward into a new cut by dropping the wing spud, raising the main one and swinging the machine around until the main spud has advanced the

FIG. 248.—SUCTION DREDGE ON PUGET SOUND.

required amount, when it is again dropped and the wing spud pulled up. Where the cut is a shallow one a leverman must watch carefully about keeping the dredge moved up, or a very great amount of waste will occur in digging too long a time or too deep in one place. The amount to be dredged is often only half of what is absolutely necessary to take out to get the lightest cut possible. The ladder is operated through a well or recess in the front end of the hull; these ladders are constructed up to 70 feet or more in length, and a 70-foot ladder will operate at a steep enough angle to make a cut to a depth of 40 or 50 feet below the water surface. The ladder can be constructed of plate girders braced together where the dredge

is to be used in still water, and where there is slight current, but if the dredge is to be used in moderate or swift current, the ladder should be made of a latticed style of trussing, which will not offer so much obstruction to the current. The cutter shown in Fig.

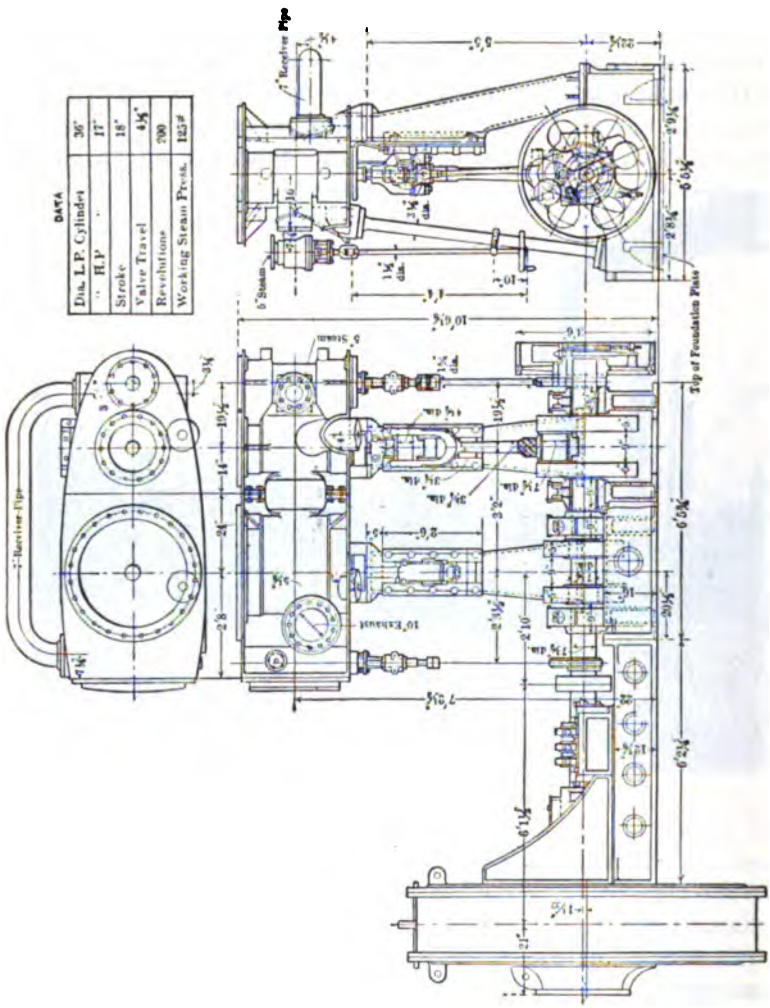


Fig. 249.—Compound Dredge Engine and Pump.

250 is of the round-nose type and best suited to soft material, but usually the ordinary inward-delivery cage cutter, Fig. 251, is to be preferred, especially for the compact and harder materials. The dredging pump is, preferably, of the type where the face of the pump can be easily removed, and new steel lining put in from time to time,

as gritty material cuts it out. For a 20-inch pump the runner should be from 5 feet 6 inches to 6 feet 6 inches in diameter, and should be run at about 180 to 220 revolutions, or so as to give a peripheral speed of about 3800 feet per minute. This will give a velocity of discharge on a short pipe line of 20 or 25 feet per second, and on a

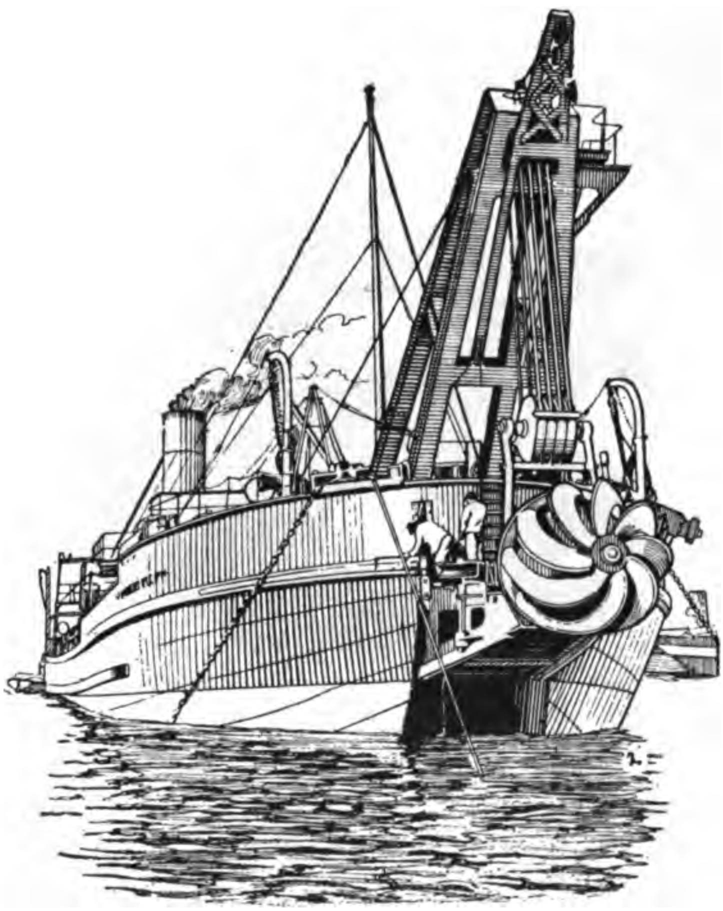


FIG. 250.—ROUND-NOSE CUTTER ON SUCTION BAR DREDGE.

long pipe line of from 15 to 18 feet per second. The pump must be firmly bracketed to the main bed connecting under the engine, and the base should be spread enough, or have a sub-base wide enough, to prevent undue vibration of the machinery. The pump and engine in Fig. 247 are placed fore and aft in the hull, but in the

writer's opinion they are much better located thwartships, as the vibrations are less, and repairs more easily taken care of, especially the relining of the pump. The engines should not be less than $17 \times 38 \times 20$-inch stroke, giving a horse-power of from 600 to 800, owing to the conditions under which they are operated. The thrust of the pump should be taken up by a regular marine thrust bearing. The cutter engines should be at least double 10×12, geared on to the cutter shaft through the axis of the suction pipe in the hinge of the ladder by beveled gearing; the whole plant to be operated condensing, with independent surface condenser, air-pump and centrifugal circulating pump. The feed-water pumps should be in duplicate, and of the outside-packed plunger type. The electric-light

FIG. 251.—SUCTION DREDGE CUTTER. CAGE TYPE.

plant is usually of a direct-connected type, and sufficient to supply lights to the entire dredge, and to a search-light of 3000 c.p.

The boiler plant consists of four Scotch marine boilers, to give 200 pounds of steam. The ordinary marine boilers are of a size based on $2\frac{1}{2}$ or 3 square feet of heating surface per horse-power, but the author prefers to figure up the entire horse-power required for operating the dredge, and to put in the Scotch boilers on the basis of 6 square feet of heating surface per horse-power, so as to have them amply large. Boilers of the Heine type are usually to be preferred, although it would then be well to allow 7 or 8 feet of heating surface per horse-power. Dredge boilers of this kind are shown in Fig. 252.

The hull of this machine is of steel throughout, with three transverse water-tight bulkheads, and stiffened by two fore-and-aft bulk-

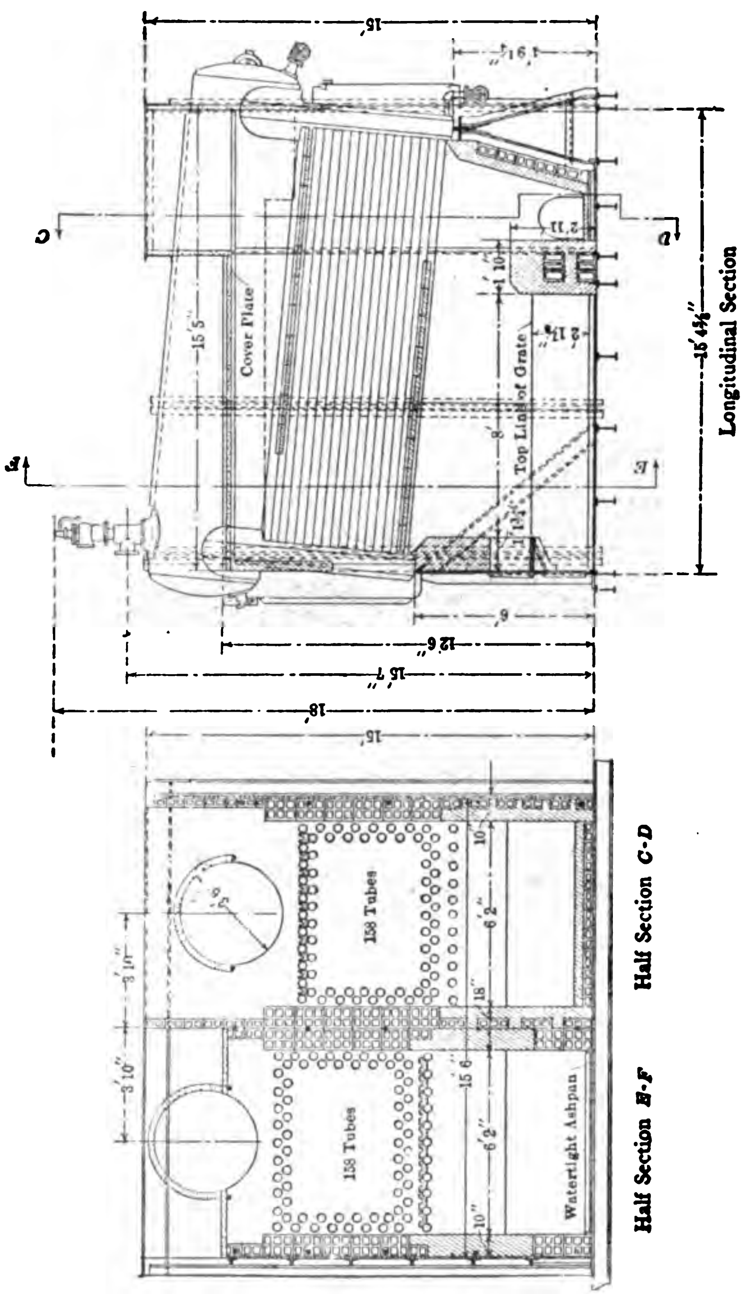


FIG. 252.—HEINE DREDGE BOILERS.

heads running the full length of the hull. The hull of the dredge *King Edward*, for the Department of Public Works, of British Columbia, was constructed with a steel frame, Fig. 253, and sheathed with plank bolted on, and calked as is usual for a wooden hull. The hull, Fig. 247, is 40×150×10 feet 6 inches deep, although many hulls for 20-inch dredges are constructed as small as 38×135×9 or 10 feet deep, especially where the quarters for the crew are left off. Where the hull is constructed of wood, it may be built either with heavy side timbers, or with frames planked. Solid transverse and longitudinal bulkheads may be used, or steel trusses thoroughly

FIG. 253.—STEEL FRAME HULL FOR DREDGE "KING EDWARD."

fastened to the timber work substituted. The discharge pipe through the hull should be of heavy cast iron, and theoretically a greater efficiency can be obtained from the pump by making it about 22 inches in diameter. This diameter can be carried through the floating pipe line if desired, or reduction made to 20 inches just outside of the hull.

The connection from the stern of the dredge to the floating pipe line is frequently made with a ball-and-socket joint, although the author prefers to use a 10-ply rubber connection 30 inches in length. The pontoons for carrying the floating pipe line in 30-foot sections

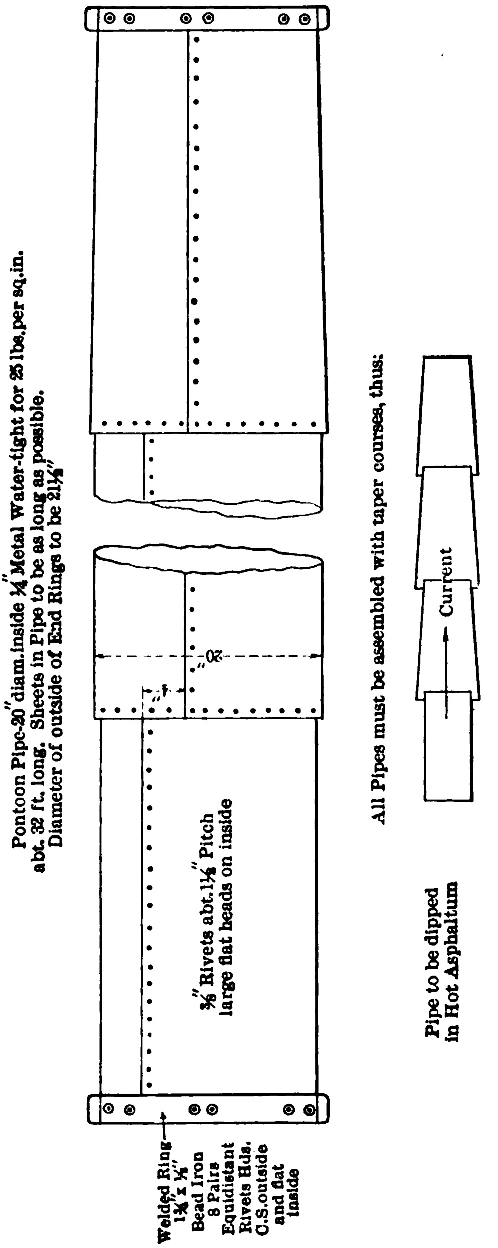


FIG. 254.—PONTOON DISCHARGE PIPE.

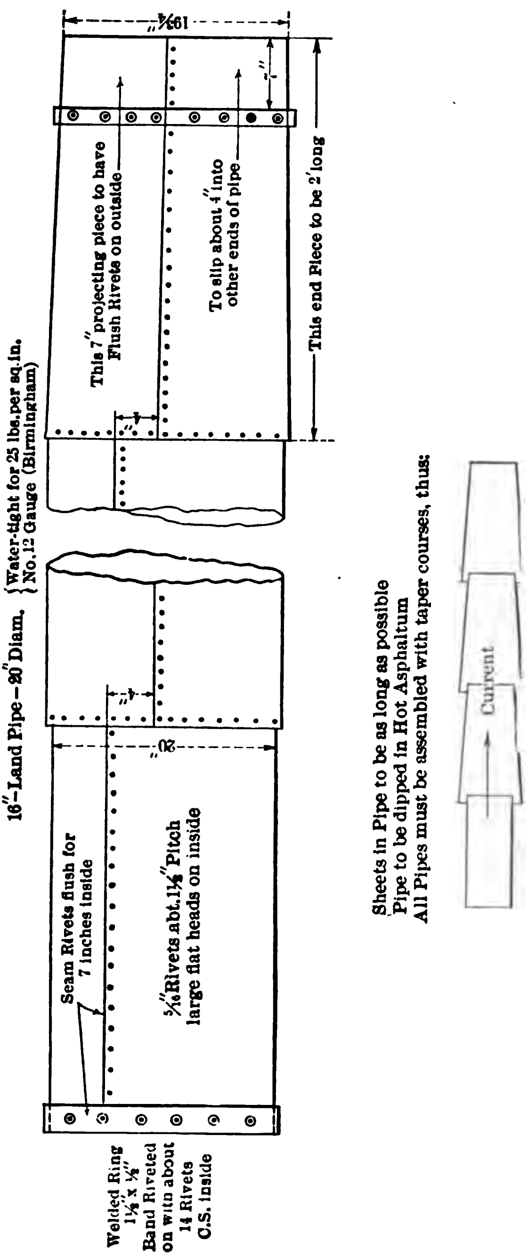


Fig. 255.—Shore Discharge Pipe.

are 10×20×2.5 feet in size. This pipe is shown in detail in Fig. 254, and is connected together with 24-inch rubber sleeves fastened on with two clamps at each end. Lap-welded pipe is often employed. Derrick scows must be provided to carry a long section pipe connecting to the shore, and the shore pipe in 16-foot lengths constructed as shown in Fig. 255, telescoping together. Sections of curved pipe with long radius should be provided to change the

FIG. 256.—SUCTION DREDGE MAKING A FILL.

direction of the pipe line on shore. Specially constructed valves can be used for changing from one point of discharge to another. The end of the discharge pipe is shown in Fig. 256, and a small platform of plank should be laid under the end of the pipe to receive the force of the stream, and sand-bags used to direct the flow as desired.

The loss of head in 20-inch pipe line approximates roughly 4 feet for each 100 feet length of pipe line, as shown by Table XXXV;

and at varying distances, owing to the conditions of the particular work in hand, it becomes necessary to provide booster pumps in the pipe line, and by this means material can be easily pumped upward of two miles.

Should it become necessary for the engineer to employ a suction dredge on foundation work, it will be well to make a contract with a reliable firm for the work at so much per cubic yard, or in case the work is such that it cannot be handled in this way, at so much per day for the operation of the dredge and equipment. Only on very

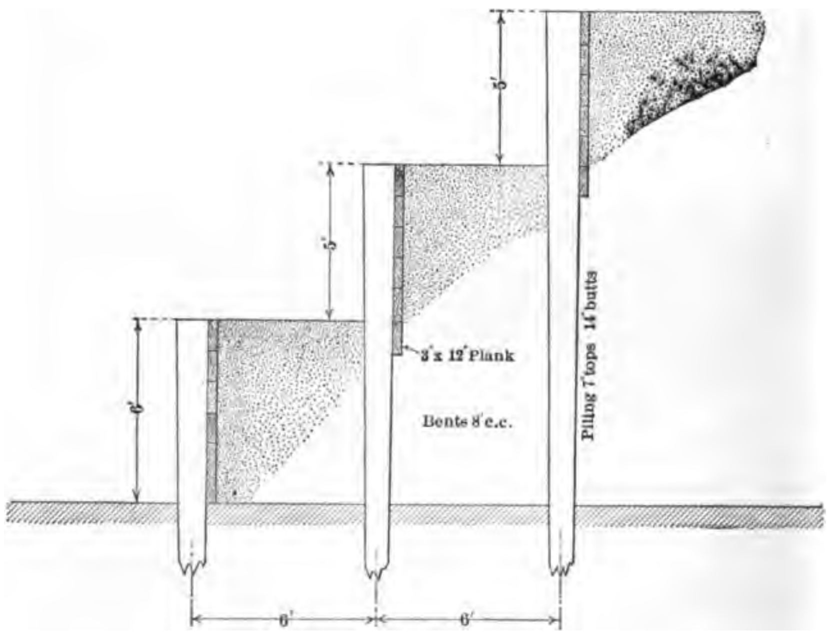


FIG. 257.—PILE AND PLANK BULKHEAD.*

extensive harbor work or river improvement would it be economical to construct such a dredging plant, as the cost of a 20-inch outfit will run from $130,000 to $150,000 for the plant complete.

The fill can be carried in any desired direction by the use of sheer boards, and can be retained by pile and plank bulkheads, Fig. 257, in case there is not too much mud in the material. In case the mud causes trouble by breaking out under the bulkhead, sand-bags can be used to stop the holes, or a layer of brush placed in the first instance underneath the bottom layer of plank. In filling large portions of the 1200 acres of the Elliott Bay Tide Flats at Seattle,

* High piles to have timber deadmen or wire ropes to anchor piles.

TABLE XXXV.—DISCHARGE LARGE PIPES. GALLONS PER MINUTE

Friction Head in Feet per 100 Feet of Pipe. Velocity Feet per Second.

Diam. Pipe.	12 Inch.		15 Inch.		18 Inch.		20 Inch.		22 Inch.		24 Inch.		30 Inch.		Diam. Pipe.
Velocity.	Capacity.	Friction.	Capacity.	Friction.	Capacity.	Friction.	Capacity.	Friction.	Capacity.	Friction.	Capacity.	Friction.	Capacity.	Friction.	Velocity.
5	1,765	0.87	2,755	0.68	3,965	0.57	4,900	0.51	5,925	0.47	7,050	0.43	11,015	0.34	5
10	3,525	3.17	5,510	2.50	7.930	2.07	9,970	1.87	11,850	1.70	14,100	1.55	22,030	1.24	10
15	5,290	6.89	8,260	5.40	11,900	4.50	14,690	4.05	17,770	3.68	21,150	3.37	33,045	2.70	15
20	7,050	11.33	11,020	9.10	15,860	7.19	19,580	6.80	23,700	6.18	28,200	5.67	44,060	4.53	20
25	8,815	16.35	13,770	13.4	19,830	11.0	24,480	10.13	29,620	8.90	35,250	8.40	55,075	6.40	25
30	10,580	22.10	16,520	17.8	23,800	14.6	29,380	13.4	35,540	11.95	42,300	11.20	66,090	8.40	30

Output in cubic yards of dredge in light sand on 1000 ft. pipe line in 20 hours running time per day will be about one-fourth of figures given for capacity in gallons per minute for clean water,

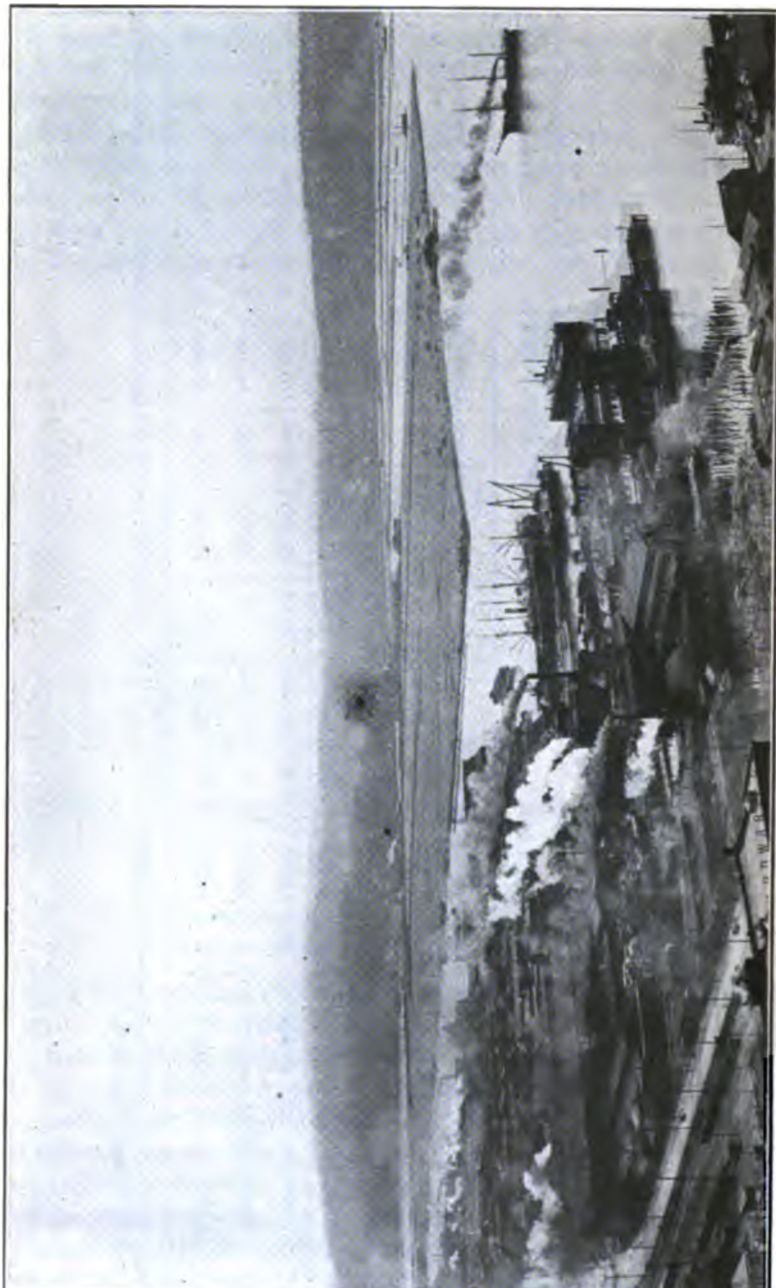


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FIG. 258.—ELLIOTT BAY TIDE FLAT FILL, SEATTLE.

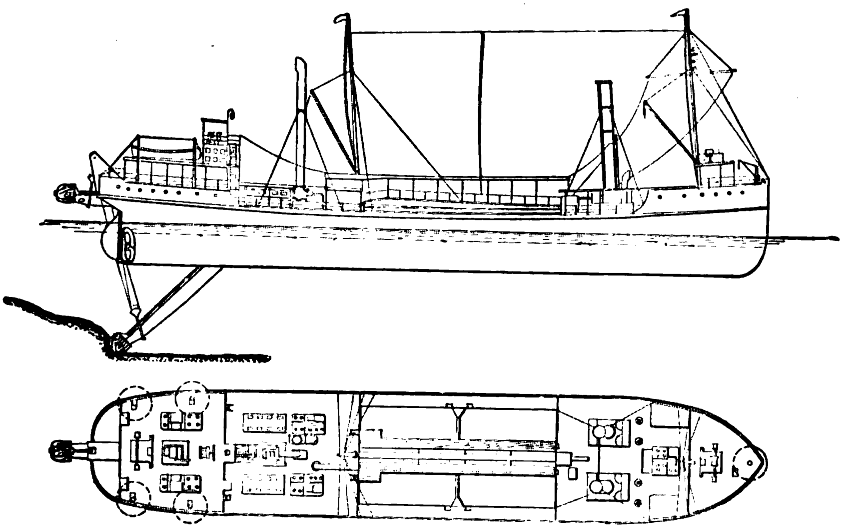


FIG. 259.—HOPPER DREDGE "MIGUEL CALMON."

FIG. 260.—EGYPTIAN SUCTION DREDGE "ABOUKIR."

Fig. 258, the author used the pile-and-plank bulkheads for interior locations, and the bulkhead for the outside was made with fir brush running from 12 to 20 feet in length, with 4-inch butts and under. Two rows of piling were driven where the bulkhead was to be placed, and the brush piled in, butts outward and on a one to one slope. This was built up in layers to keep pace with the elevation of the fill, and practically none of the material was lost.

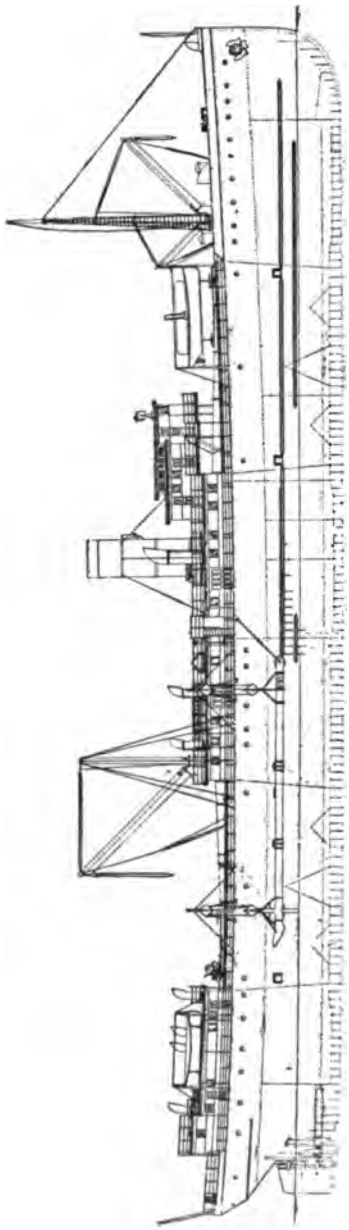


FIG. 261.—SEA-GOING DREDGE "GALVESTON."

The rough surface of the bulkhead formed by the butts of the brush works admirably in rough water, the waves breaking up against it and falling dead. This work, comprising upward of 20,000,000 cubic yards, was carried out at a figure of 18.4 cents per cubic yard in place.

The dredge *Miguel Calmon*, shown in Fig. 259, fitted with ladders and cutters, discharges into a hopper, and this hopper can also be emptied by the pump of the dredge. The dredge *Aboukir* (Fig. 260) is arranged to dredge from the bottom or to pump out scows alongside, and deliver the material on shore. Large sea-going dredges of the hopper type are well illustrated by the *Galveston* (Fig. 261), described by T. M. Cornbrooks in a paper before the American Society of Naval Architects as follows: "During the past five years the engineers' department, United States army, have contracted for a number of sea-going suction dredges. These have

been of varying sizes, ranging from 166 feet to 300 feet long, the capacities of bins ranging from 1000 to 3000 cubic yards. As the design of all these dredges has been similar, the description of one will apply to all. The latest dredge is the *Galveston*, just completed at the works of Maryland Steel Company, Sparrow's Point, Maryland. As will be seen by the plate the dredged material is carried in two large bins, one forward and one aft of the machinery space.

" In operating, the dredge is kept moving forward at a speed of about 6 knots, with the suction pipes dragging on the bottom. The material is sucked up by 20-inch centrifugal pumps (Fig. 249) and discharged into bins, through pipes and distributing chutes on top of bins. By means of gates in the bottom and sides of these chutes it is possible to distribute the material evenly. As the bin fills the water is drained off by overflow through the sides. When the bins are filled the dredge proceeds to the dumping grounds and, opening the gates in the bottom, drops the material. The gates are operated by means of a double-cylinder vertical engine through worms and fixed nut on vertical rods.

" The officers and crew have commodious quarters in houses on deck and on the lower deck forward and aft.

" The propelling machinery consists of two compound engines, steam for which is furnished by four single-ended boilers.

" In trying these dredges, we have discovered that a very large rudder is necessary. This we ascribe to the shafts being so far outboard and the extreme fullness of the after body."

No further discussion will be given of sea-going suction dredges, as they can seldom be employed in any way in foundation work.

CHAPTER XIX

TUGBOATS AND SCOWS

THE use of tugboats and scows on large foundation works has been referred to in various other chapters in this book, but as the published information on the subject is very fragmentary and scattered, it will be of value to collect the data together, and give such general information as the engineer should be possessed of to properly handle and take care of such plant.

While it is often possible on large work to save money by constructing such equipment, and afterward getting as much out of it in salvage as possible, it is very often more convenient to buy launches, tugboats, and scows second-hand, and save considerable first outlay; although to do this intelligently the engineer must have sufficient knowledge of the subject to place a proper valuation upon such floating plant.

The economy of small gasoline launches for running around over the work, and carrying small supplies, makes one or more boats of this kind an absolute necessity on large work. If constructed sufficiently large and heavy, they will be of service in taking the men out to the work, and in handling small scows, pile-drivers, piling, or anything else in the way of towing that comes within the capacity of the launch. For the purpose of simply superintending the work, or carrying small supplies, a launch from 20 to 25 feet in length, with a 6- or 8-horse-power gasoline engine giving a speed of eight or ten miles per hour will be all that is required. Such a launch may often be picked up for a few hundred dollars, or can be constructed new for $500 or $600.

The launch shown in Fig. 262 is of a very handy size, being 21 feet 9 inches in length, and 4 feet 11 inches beam. The boat has a raised turtle deck for 9 feet back of the bow, being carried farther back than is shown in the plan, in order to cover the gasoline motor, and still give room enough to work underneath. The engine is a 3-cylinder 12-horse-power Ferro, operating a 15×20-inch 3-bladed Hyde propeller up to 970 revolutions per minute, which makes it

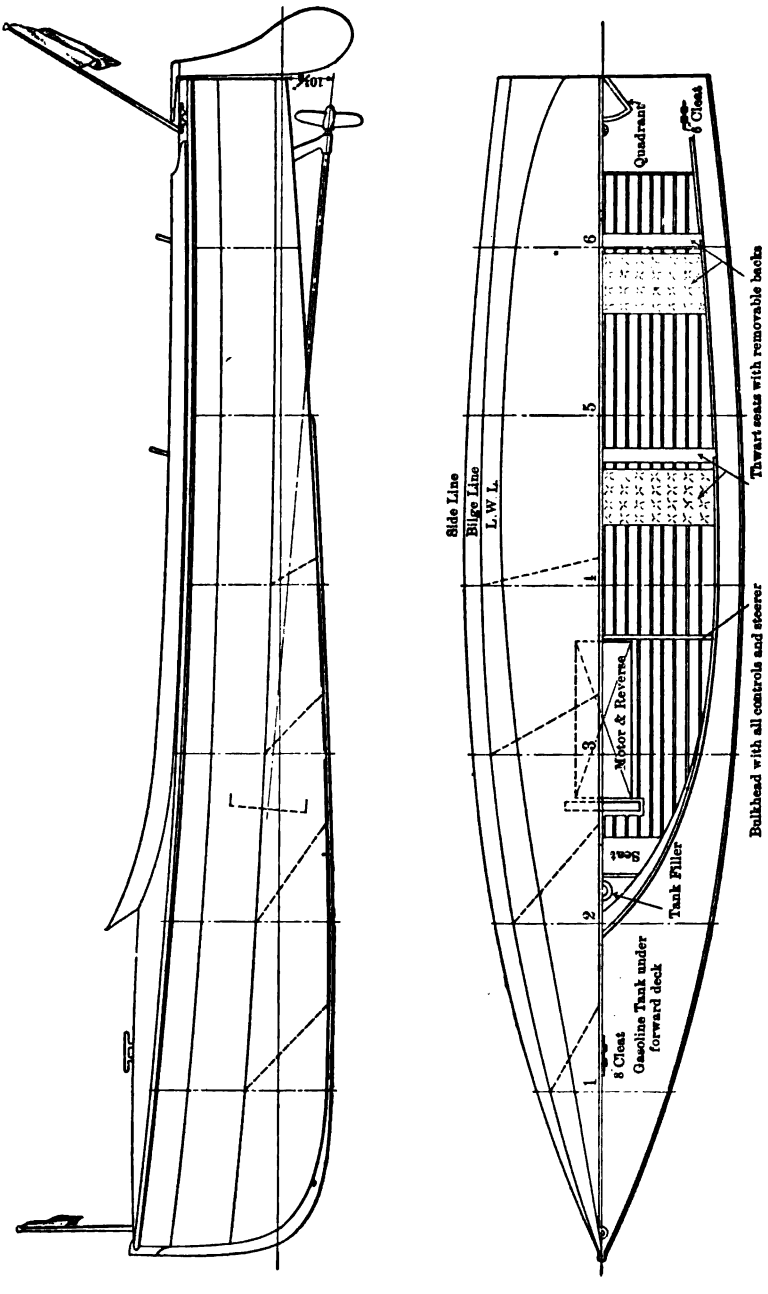


FIG. 262.—TWENTY-ONE FOOT V-BOTTOM LAUNCH.

possible to run the boat a little better than 15 miles per hour, this being fast enough for all practical purposes. The equipment of the boat is probably the most complete of any of her size on the Pacific coast, including as it does the motor with a Bosch dual magneto, rear starting device, clutch, Gray-Hawley whistle outfit (which also furnishes pressure to the gasoline feed), two gasoline tanks with gage to show the level of the fuel in the tanks, water separator on the gasoline line; electric lights and running lights, 14-inch search-light, cigar lighter (all supplied by Edison new-type storage battery); Rochester auto wheel and controls, power bilge pump, Ever-ready tachometer giving R.P.M. of engine on bulkhead, suction ventilator on deck, Kenyon cushions, glass wind shield and spray wings, 2-inch Dirigo compass, fire extinguisher, complete set of tools in racks to fit, rubber matting on floors, three anchors and lines, and 10-foot tender. All fittings are of bronze, brass and copper. The outfit of course would not need to be so elaborate or expensive for ordinary work.

Gasoline tugs have become very popular for small towing, inasmuch as they can be handled by one licensed man acting as captain and engineer, when the levers for controlling the engine are carried into the pilot house. Where there is considerable towing to do, he will need the assistance of one or two deckhands. The gasoline tug *Ohio*, Fig. 263, constructed for the author on the rock work described in Chapter XXIV, is the handiest boat of the kind that could be desired around contract work. The line drawings of this tug are shown in Fig. 264, and the general dimensions of the boat are 36 feet in length, 10 feet beam, and 5 feet depth of hold, with a draft of $3\frac{1}{2}$ feet. It is operated by a 32-horse-power gasoline engine, and is heavy enough to handle 400-ton loaded scows around the work, and will handle a 250-ton scow on a 20- or 30-mile tow. The hull has a high bow and freeboard forward, and raised deck over the engine room, which is forward, thus giving head room. The house is large enough to shelter 8 or 10 workmen in bad weather, and there is a roomy afterdeck, with the tow-bitts placed just aft the house. With heavy frames and planking the boat would readily carry a 50-horse-power gasoline engine, adding greatly to the towing capacity. Running light the boat will make 10 or 12 miles per hour. The cost complete, ready for operation, was approximately $2500.

The 55-foot gasoline tug shown in Figs. 265 and 266 is described in the *Pacific Motor Boat:* " A substantial-looking work boat, the *Bamberton*, recently built to the designs of Messrs. Morris, Bulkeley & Halliday at the yards of the Hinton Electric Company, Victoria,

for the Portland Cement Construction Co., Ltd., of Bamberton, Tod Inlet, Vancouver Island, has just been placed in commission by that company. Her leading dimensions are: Length over all, 55 feet; length on water-line, 50 feet; beam, 12 feet 6 inches; depth, molded, 5 feet 6 inches; and draft, 4 feet 6 inches.

"The vessel is required for towing scows from the quarries to the cement works, a distance of about 4 miles, and the waters in Tod Inlet being in some parts shallow, the draft was limited to 4 feet 6 inches.

FIG. 263.—GASOLINE TUG "OHIO."

"Referring to the plan it will be seen that the engine is located amidships in a spacious engine room between water-tight bulkheads; the accommodation for the crew of two men is under the raised deck forward and it will be noticed that it is very commodious and comfortable; abaft the engine space is provided ample room for stowing gear and stores.

"The machinery consists of an 80-horse-power Union gasoline engine, $9\frac{3}{4}$-inch bore by $10\frac{1}{2}$-inch stroke, running at 310 R.P.M., of the open cross-head type, and it is fitted with the Union Gas Engine Company's new intake pipe, by means of which it is possible to operate successfully on cheap grades of fuel. The propeller is the

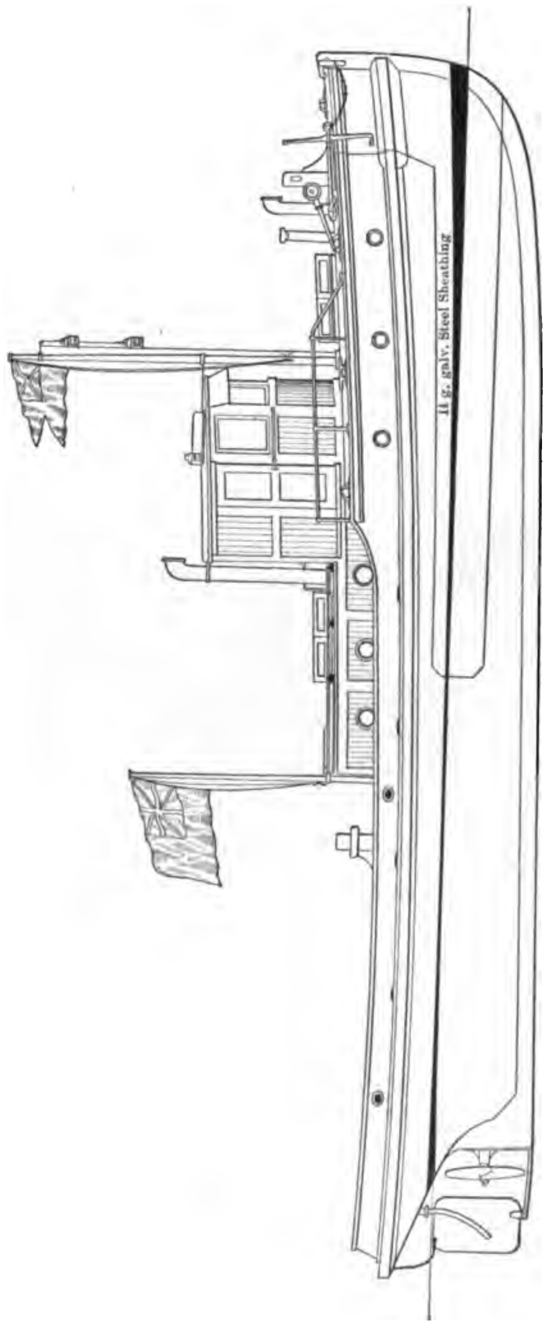


FIG. 265.—CANNERY TENDER 55 FEET LONG.

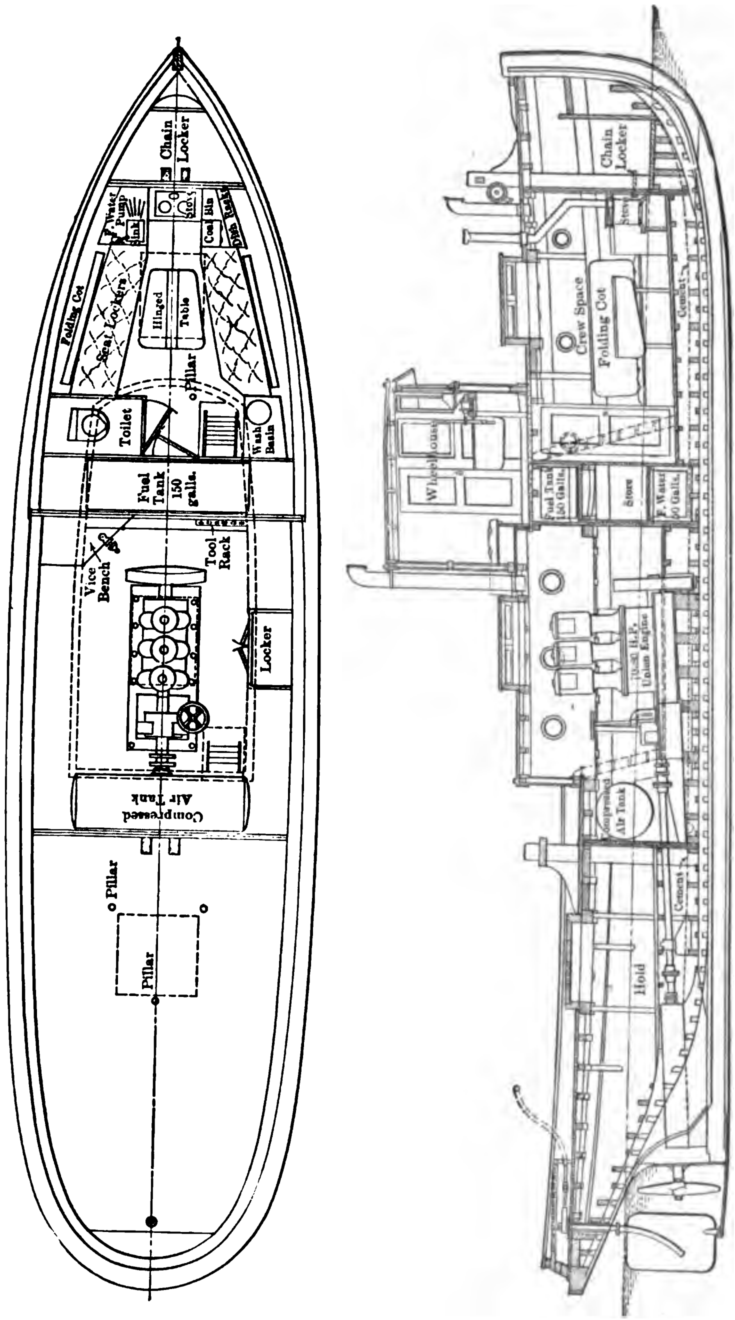


FIG. 266.—PLAN AND SECTION OF CANNERY TENDER.

well-known Hyde turbine pattern, 44 inches in diameter by 48 inches pitch; the gasoline tank is placed under the pilot house in a sealed chamber just above the deck line and is fitted with a drain leading overboard.

"Exhaustive trials have been carried out with the vessel to test her speed and seaworthiness before she was taken over by the company and all through she has more than fulfilled the expectations of the owners and their consulting naval architects, a speed of 10.7 miles per hour being obtained without forcing the engine while the vessel was running in a heavy sea."

The tugboat *Crowley No. 21* is described in the *Pacific Motor Boat* as follows: "One of the late additions to the Crowley Launch & Towboat Company is the *Crowley No. 21* and her plans will show that she was built for towing only, every feature of her design and arrangement tending toward efficiency and handiness in this respect.

"The *Crowley No. 21*, was designed by D. W. & R. Z. Dickie of San Francisco and built by W. G. Tibbets & Co., of Alameda. The features of the new law which restricts length rather than tonnage was taken advantage of, and she was given 30 inches of freeboard instead of 20 inches or less as was the case in boats built under the old 15-ton law, which makes her dry in rough weather.

"Some of the new features were a towing bitt with a 9-inch saddle in order to save expense in lines, also a cast-iron riding bit forward with a power capstan. In order to meet the requirements of the Board of Marine Underwriters, a 6-inch 'Dow' centrifugal pump was installed which is arranged to engage the flywheel and can be used to flood or pump out a vessel.

"The two main fuel tanks hold about 400 gallons of fuel each and are built according to the rules of the U. S. Boiler Inspectors and equipped with an air-pressure attachment which forces the distillate from the large tanks to the small tank in the forecastle for gravity feed.

"A low house covers the engine, and a large roomy pilot house shelters the operator and also provides a bed in case of a long tow. This is done by folding out the seat and taking out the bedding from under it. The windows, door and skylight are glazed with $\frac{5}{16}$-inch French plate glass. A complete system of ventilation is installed to prevent the accumulation of gas in any part of the hull.

"A complete auto control system is fitted on a stand in the pilot house which steers the boat and controls the engine, both the spark advance and governor being led to this point.

"The *Crowley No. 21* is of the model known as the straight stem, elliptical stern, with a chock at the bow and stern and a log rail all around. The guard is heavy and is faced with $\frac{1}{2}\times 5$-inch iron all around the boat. The dimensions are as follows:

Length over all	50′ 5″
Length from the rabbet of the stem to back of the stern post at the shaft	41′ 11½″
Beam over plank at deck	14′ 1″
Draft forward at the forefoot	3′ 10″
Draft aft at sternpost	6′ 0″
Freeboard at lowest point of sheer	2′ 4″
Sheer forward	3′ 10″
Sheer aft	10¼″
Depth of hold	5′ 2″

"The motive power of the *Crowley No. 21* is an 80-horse-power 3-cylinder Union gas engine with cylinders 9¾ inches diameter and 12-inch stroke, fitted with a patent heater on the intake pipe made by the Union Gas Engine Company of San Francisco, which gives greatest elasticity to the control of the engine; the revolutions of the engine can be reduced to 42 turns without stopping, also the engine can be run on low-grade fuel.

"The *Crowley No. 22* is 57 feet 0 inches over all, 14 feet 0 inches beam molded, and 16 feet 0 inches over the guards, with a draft aft of 6 feet 3 inches at the sternpost. Her deck line is very full forward and wide aft in order to give large deck space for passengers or freight.

"She is equipped with a 3-cylinder 100-horse-power Union engine with cylinders 12-inch bore by 15 inch-stroke, turning 300 revolutions per minute. In arrangement she is similar to No. 21."

The engines for boats of this class have reached a high state of development, and are as reliable as automobile engines. The engines of the two-cylinder type are shown in Fig. 267, and the dimensions given in Table XXXVI. Four-cylinder engines are shown in Fig. 268, and the dimensions given in Table XXXVII, which will cover as large sizes as are often used around contract work. The engine room of a twin-screw gasoline boat, constructed by the author for use on Puget Sound is shown in Fig. 269.

The selection of propellers for motor boats is one of the most important features to take care of, and is fully covered in a paper by Alfred J. C. Robertson. "The necessity of having the proper

size and design of propeller on gasoline boats is really much more important than on steamboats, as in the latter, if the propeller is

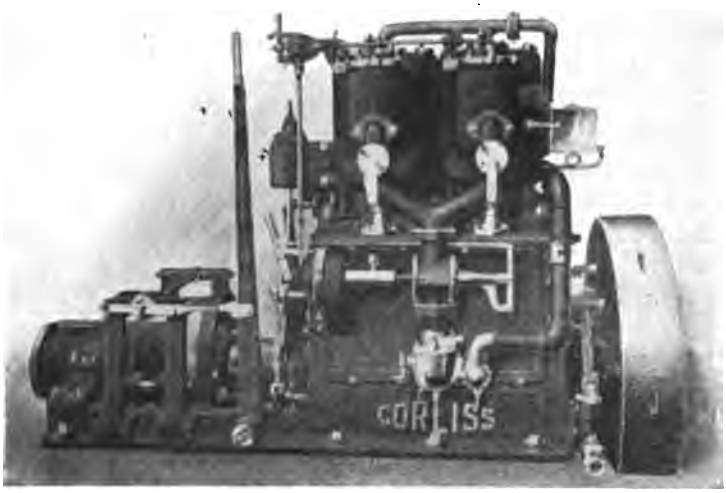


FIG. 267.—GASOLINE TUG ENGINE, TWO CYLINDER.

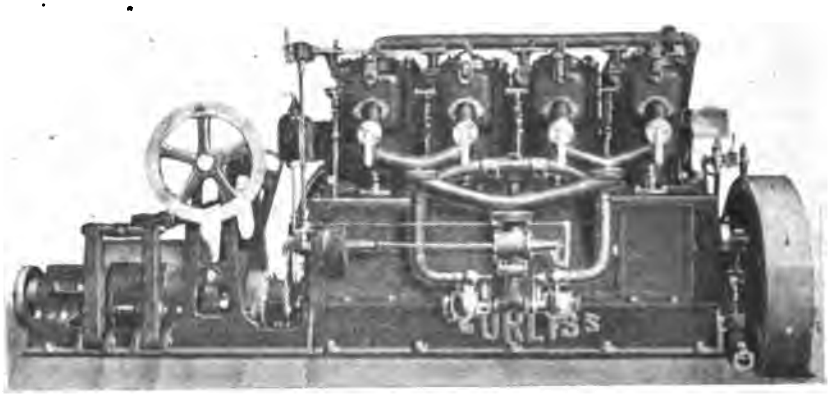


FIG. 268.—GASOLINE TUG ENGINE, FOUR CYLINDER.

too large it of course reduces the revolutions of the engine, but the steam pressure may usually be increased and the power of the

engine will be maintained, but in the former case if the revolutions are reduced the power is reduced at least as much in proportion, and with this reduction of the power of the engine the revolutions again come down, This loss in power is altogether distinct from the loss of efficiency due to a badly designed screw, a loss which may occur even when the engine maintains its full rated power.

"These two losses together make it, therefore, most important to use your engine power, that the propeller fitted should not be too

FIG. 269.—ENGINE ROOM OF TWIN-SCREW GASOLINE DISPATCH BOAT.

large or too small in diameter, and that the pitch and surface of the blades should be carefully calculated.

"There are an infinite number of ways in which a propeller could be varied, and the cost of testing these on a boat would, to say the least, be colossal, so that the most valuable information we have to-day has been obtained by testing small models in a tank, and discovering the law of comparison by applying these results to full-size propellers.

"Such tests have been carried out chiefly by D. W. Taylor, of the United States Experimental Department, and by R. E. Froude, Esq., of the British Admiralty.

"Mr. Taylor has published a large number of results of such

trials, but Mr. Froude has gone a step farther and has enunciated a law which governs the propeller results, and this law is expressed by the formula:

$$\text{T.H.P.} = \frac{D^2V^3}{144} \times \frac{B(P+21)}{P} \times \frac{S(1-.08S)}{310.9(1-S)^2}.$$

Where T.H.P. = Actual thrust of propeller expressed in horse-power;
D = Diameter of propeller in inches;
V = Speed of advance of propeller through the water;
B = Blade factor, varying with the area of blade surface;
P = Pitch ratio, that is $\frac{\text{Pitch}}{\text{Diam.}}$;
S = Effective slip ratio (*not* the usually observed slip).

"Now, though this formula of Mr. Froude's is very simple and may easily be worked out by arithmetic, unfortunately with the exception of D every term in the formula has to be modified to suit the special purpose to which it is to be applied.

"Let us see what these modifications are: T.H.P., or thrust-horse-power, expresses the actual push forward that the propeller gives the boat and is less than the brake-horse-power by:

"(1) The loss in efficiency of the propeller itself. This will be from 25 to 40 per cent.

"(2) The loss due to friction in stern bearings and thrust block.

"(3) The loss due to the suction of the propeller upon the boat. The total loss of power from these three causes amounts to from 40 to 60 per cent. of the brake horse-power, so that the T.H.P. is approximately half of the B.H.P.

"V, the usual symbol for speed in knots, in the formula refers to the advance of the propeller through the water, but this is nearly always less than the speed of the ship in knots, because when a boat is under way it draws the water after it, to a certain extent, and this wake may reduce the speed of advance of the propeller by 25 or 30 per cent. even, though usually the wake reduction is somewhat less than that.

"B, the blade factor, varies from about .10 to about .13, the former being for very narrow blades and the latter for wide, broad-tipped ones.

"P, the pitch ratio, is about 10 per cent greater than the nominal or face pitch ratio (pitch ÷ diam.) because the actual effective pitch is affected considerably by the shape of the back of the blade, a thick blade producing an increase of effective pitch.

" S, the slip ratio, is subject to the same correction as V, the speed, the actual slip being much greater than the apparent slip, due to the wake of the boat.

" For motor cruisers such as the average boat around the Coast, the corrections necessary for propeller suction, shaft friction, wake, blade width and blade thickness are practically constant, and all these corrections can be incorporated in a new formula, but we can only approximate the propeller efficiency, as this varies somewhat with variation of pitch.

" Approximating this propeller efficiency and including the suitable corrections for wake, etc., our formula will read:

$$\text{B.H.P.} = \frac{D^2V^3}{372000} \times \frac{P+21}{P} \times \frac{S(1-.08S)}{(1-S)^2}.$$

" S, the slip percentage, being obtained by the formula:

$$1-S=\frac{973V}{RPD}, \quad R \text{ being revolutions per minute.}$$

" Let us work out a couple of examples from these formulæ:

" Suppose we have a propeller of 24 inches diameter, with a pitch ratio of .80, which gives an effective pitch of 19.2 inches with three blades of average width, what horse-power would it require to drive it 750 revolutions at a ship's speed of $8\frac{1}{2}$ knots.

" First, let us find out what the slip would be:

$$1-S=\frac{973 \times 8.5}{750 \times .8 \times 24}=.575 \text{ and } S=.425.$$

Then,

$$\text{B.H.P.} = \frac{24^2 \times 8.5^3}{372000} \times \frac{21.8}{.8} \times .425 \frac{(1-.034)}{.575^2}.$$

Therefore,

$$\text{B.H.P.} = \frac{576 \times 612 \times 21.8 \times .425 \times .966}{372000 \times .8 \times .575 \times .575} = 32.06.$$

If we take the same propeller with a pitch ratio of .60 we will have

$$1-S=\frac{973 \times 8.5}{750 \times .6 \times 24}=.767 \text{ and } S=.233.$$

Then

$$\text{B.H.P.} = \frac{576 \times 612 \times 21.6 \times .23 \times .982}{372000 \times .6 \times .767 \times .767} = 13.10,$$

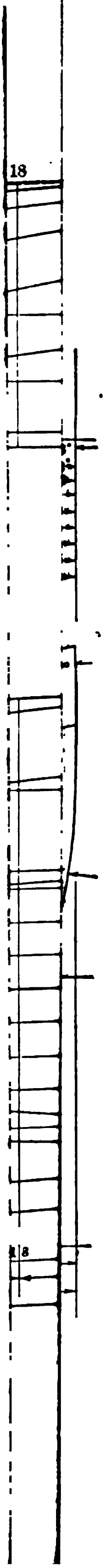
thus showing that any change in pitch ratio will make quite a serious difference in the power of the propeller.

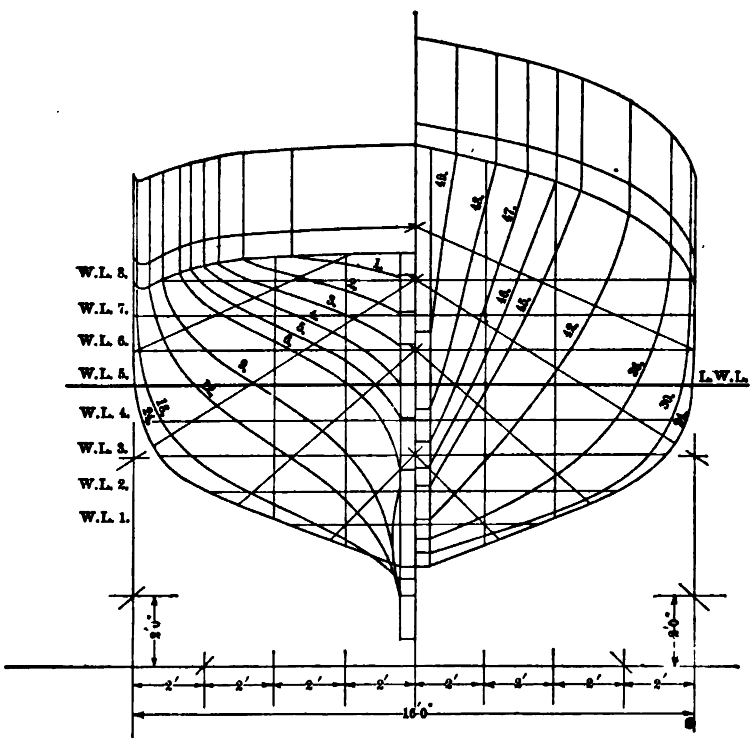
"While this formula will give the power required to drive any known propeller at a required speed, it is of great value also in that it indicates how changing diameter,. pitch, etc., changes the power absorbed by the propeller."

Where it is necessary to use larger towboats than 50 horse-power, it is very often advisable or necessary to use a steam tug. The *Miami*, used by the author for eight or ten years on Puget Sound, is shown in Fig. 270, and is shown in plan in Fig. 271. This boat is equipped with a fore-and-aft compound engine $10\times 20\times 10\frac{1}{2}$ inch stroke, giving, with 150 pounds boiler steam pressure about 60 horse-

FIG. 270.—STEAM TUG "MIAMI."

power. The auxiliaries consist of air-pump, circulating pump, hot well, boiler-feed pump, and an outside pipe condenser shown in Fig. 272. The pipe used is galvanized. and has to be renewed every year or so, so it is better to employ an inside surface condenser of the type described in the succeeding pages. The boiler is a Clyde boiler 72 inches in diameter, and 9 feet in length, with approximately 540 square feet of heating surface. A water-tube or pipe boiler sufficient to furnish steam for the engine on this tug would have a heating surface of 840 square feet, and a grate area of 24.35 square feet. Such a boiler would carry 250 pounds of steam, and would weigh 13,000 pounds, or with the water 14,227 pounds, and will cost about $2 per square foot of heating surface. A tugboat of this





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kind handles, on a 30-mile tow, 3 loaded scows carrying 300 tons each, and returns the same distance with 4 light scows, making the round trip in twenty-four hours, taking account of tides.

The boat running light will make about 8 or 9 miles per hour, or from 3 to 5 miles per hour with a tow, carries 6 tons of coal, 1200 gallons of water, and has a coal consumption of about 200 pounds per hour. Such a boat can very often be purchased for about $7500,

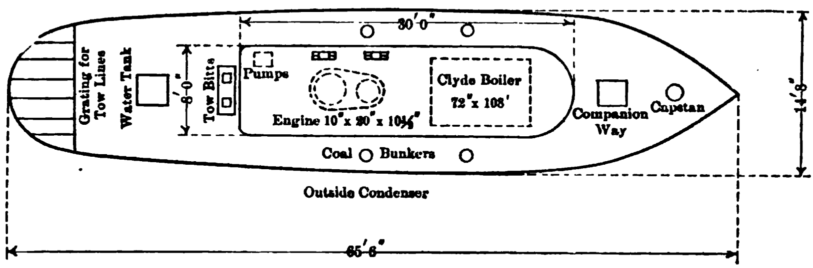


FIG. 271.—PLAN OF STEAM TUG "MIAMI."

and would cost new, fully equipped, about $11,000. The line drawings for a tugboat hull 75 feet long of extra heavy construction are shown in Fig. 273.

The building of timber hulls of Douglas fir is carried on very extensively in the Puget Sound country, and a steam schooner hull built by the author for use in Pacific Coast trade is shown in Fig. 274 with the frames all erected and in Fig. 275 for launching. The

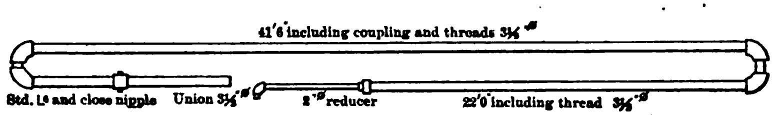


FIG. 272.—OUTSIDE CONDENSER.

cost of rigging up a shipyard for such work is comparatively small, and the same outfit can be used in building dredge hulls (Fig. 284), scows or tugs.

The two new tugs built for the Department of Wharves, Docks and Ferries of the City of Philadelphia have been delivered by the builder, the Waters-Colver Company, West New Brighton, Staten Island. These tugs were built complete, including the engines and machinery. They were delivered to Philadelphia the opening day of the inside route between Philadelphia and New York. The *Kensington* is the larger of the two tugs, being 65 tons, built of wood, 81 feet long,

FIG. 274.—FRAME OF TIMBER HULL IN SHIPYARD.

FIG. 275.—TIMBER HULL READY FOR LAUNCHING.

20 feet breadth and 9 feet depth. She is equipped with a compound fore-and-aft engine $12 \times 26 \times 18$ inches, has one Scotch boiler

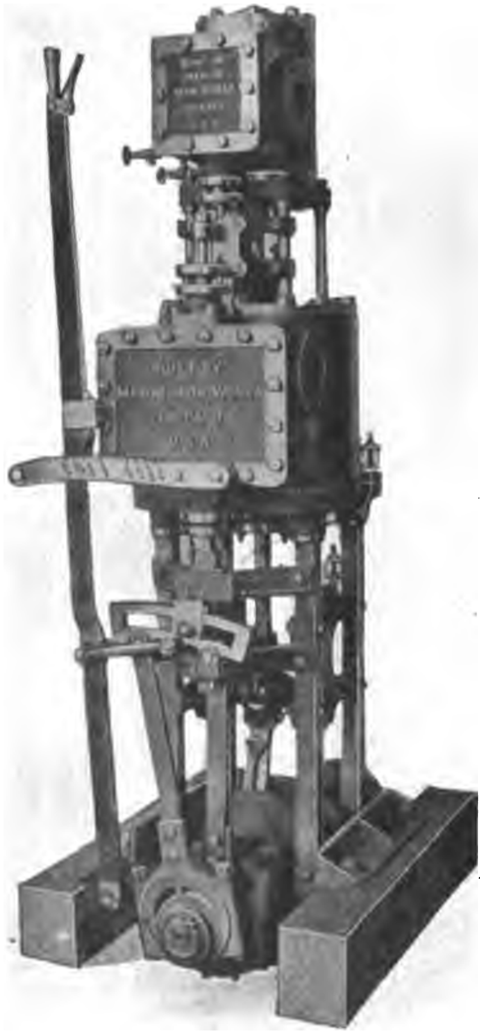


FIG. 276.—STEEPLE COMPOUND TUGBOAT ENGINE.

10 feet 6 inches by 10 feet. The *Southwark* is a 42-ton tug constructed of wood, 66 feet long, 16 feet breadth and 7 feet depth of

FIG. 277.—FORE-AND-AFT COMPOUND TUGBOAT ENGINE

hold. She is equipped with a single engine 14×14 inches, with one marine leg type boiler 5 feet 6 inches by 10 feet. On the day of the trial trip, the *Kensington* developed a speed of 13.9 miles per hour, much exceeding the contract requirement of 10.5 miles.

Should it be desired to equip a small tugboat with a compound steam engine, and the space be limited, a steeple compound, as shown in Fig. 276, can be used. This is a vertical tandem compound, with the high-pressure cylinder vertically over the low-pressure cylinder. The one shown in the illustration is 7×14×8-inch stroke, and with 170 pounds of steam would develop about 30 horse-power. The ordinary tugboat engine is of the fore-and-aft type, the one shown in Fig. 277 being 12×25×16-inch stroke, and with 170 pounds steam pressure will develop about 150 horse-power. The horse-power of compound marine engines may be calculated from Seaton's formula, giving the estimated horse-power:

$$\text{E.H.P.} = \frac{D^2 \times \sqrt{P} \times R \times S}{8500}.$$

D = Diameter of L. P. cylinder in inches.
P = Absolute boiler pressure in pounds.
R = Revolutions per minute.
S = Stroke in feet.

The boiler for such a tug can be of the water-tube type previously described, or preferably, if it is desired to have the very best, of the regular Scotch marine design, Fig. 278. This drawing shows clearly the method of construction, the combustion chamber, and the water back. The details of Scotch marine boilers from 6 feet in length to 10 feet in length are given in Table XXXVIII. The horse-power of boilers of this kind is usually figured on the basis of from $2\frac{1}{2}$ to 3 square feet of heating surface per horse-power, although the author prefers to use not less than 4 square feet per horse-power in order to have the boiler plenty large, as nothing is more exasperating than to be lacking in steam in an emergency. Where oil fuel is to be used the grate bars are omitted, and the bottom of the furnace lined with firebrick, and a back wall of the same kind of firebrick built up to such a height as is necessary at the back of the furnace to insure proper combustion.

To arrive quickly at the approximate heating surface of a small Scotch marine boiler of good design, the surface of tubes may be figured up and 25 per cent added for the balance of the heating sur-

face in the boiler. For medium-sized boilers 30 per cent. should be added. The total heating surface in a well-designed Scotch boiler should run approximately $2\frac{1}{10}$ square feet of each cubic foot of the

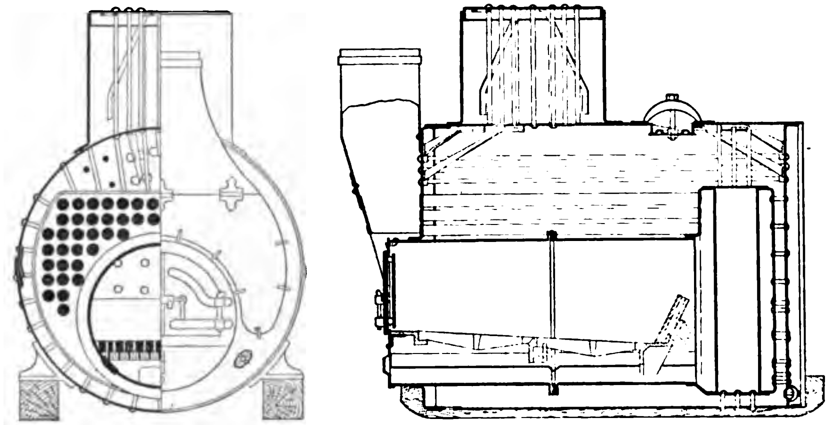


FIG. 278.—SCOTCH MARINE BOILER.

size of the boiler. The weight per cubic foot will run about 60 pounds, and the weight of the water will be about 30 pounds per cubic foot of the entire size of the boiler.

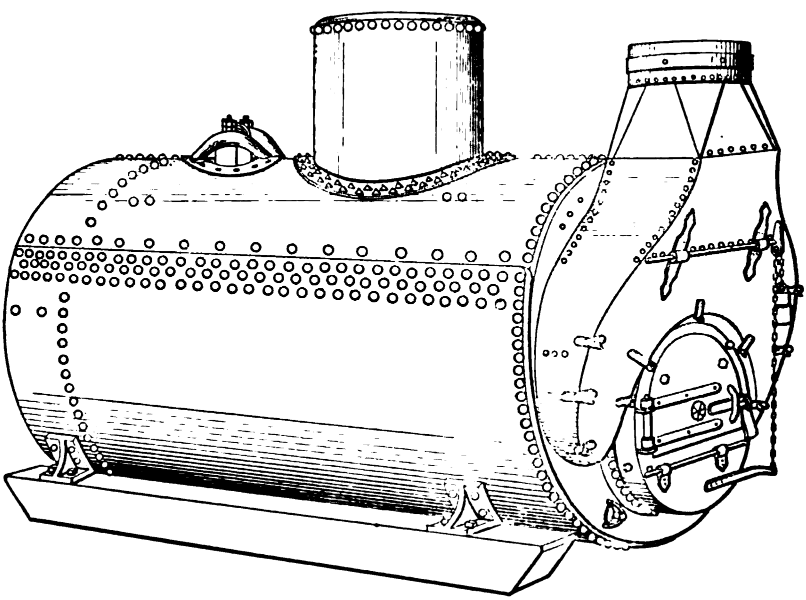


FIG. 279.—CLYDE OR DRY-BACK SCOTCH BOILER.

The Clyde boiler, or dry-back Scotch boiler, is shown in Fig. 279, and the data for such boilers from about 4 to 10 feet is given in Table XXXIX. The water space at the back of the ordinary Scotch boiler is replaced by a combustion chamber with a removable back, which has an asbestos or firebrick lining.

Tugboats on fresh water may be operated non-condensing if economy is not fully to be considered, but where economy is taken account of or on salt water, such machinery is of the condensing type, and an independent surface condenser with air-pump is shown

FIG. 280.—SURFACE CONDENSER AND AIR-PUMP FOR TUGBOAT.

in Fig. 280. A hot well of galvanized iron must be provided and a centrifugal circulating pump, Fig. 281, to supply the necessary cooling water. Such a pump should also be connected to the bilge to keep the hold dry, and to supplement the ordinary bilge-pump connected to the front end of the engine shaft.

The equipment of launches, gasoline tugs, and steam tugs must be in accordance with the requirements of the Government Steamboat Inspection Service, and the crews of the size required by law. The number of men called for on a tugboat similar to the *Miami*, Fig. 270, is for a single crew, a captain, engineer, fireman, and two deck-hands, when the boat is not operating more than thirteen

hours per day. Boats of this size are usually equipped with a galley for feeding the crew, and in such a case the cook is counted as one of the deck-hands. Where the boat is to be operated twenty-four hours per day, a mate and assistant engineer must be added to the crew, making a total of seven men. The ordinary rate of hire for

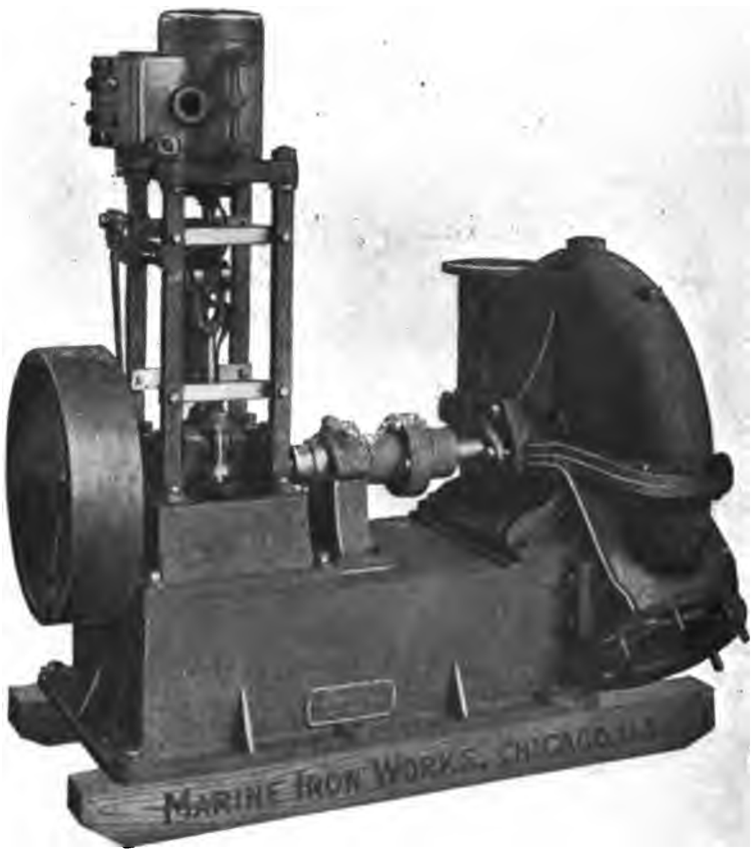


FIG. 281.—CIRCULATING PUMP FOR TUGBOAT.

such a boat on Puget Sound averages about $35 per day, and will cost for operation about $25 per day exclusive of repair and depreciation, which will be not less than about 15 per cent. per annum on the new value of the boat.

Hulls, boilers, and machinery must be kept in the best of condi-

tion to insure steady operation and to pass Government inspection, which is very severe, and at least once a year, just previous to inspection and again in six months, the hull should be pulled out on marine ways and thoroughly overhauled and given two coats of copper paint. The hull above water and the upper works must be kept in first-class repair and well painted, or in " ship shape " at all times.

The scows or lighters used on contract work are of various sizes, as may be seen by reference to the list of equipment in Chapters X and XI. A scow such as was used on the work described in Chapter X, 60 feet long, 22 feet beam, and 5 feet 6 inches deep, is shown in Fig. 282, which is a copy of the working drawings used in the construction of a number of scows of this size, at a cost of about $1650 each, although they can be built more cheaply by cutting down on fastenings and calking. Instead of the longitudinal cross-bracing shown, two solid bulkheads of 6-inch timbers drift-bolted together can be substituted if desired to spend more money in making the scows heavier and more lasting. In place of the transverse cross-bracing natural ship knees may be used.

The material required to construct a scow of this size is shown approximately in the following list:

BILL OF LUMBER

2	6″	×16″	60′ 0″	Strake No. 1	S. 1 S.	C. S.
2	6	×14	58 0	" 2	"	"
2	6	×14	54 0	" 3	"	"
2	8	×16	50′ 0	" 4	"	"
2	12	×18	22 0	Transoms	S. 1 S., 1 E.	
5	6	×10	22 0	Deck Beams	Rgh.	
5	6	×10	40 0	Deck Beams	"	
10	6	×10	50 0	Keelsons	"	
12	6	×10	22 0	Cross Beams	"	
10	6	×10	8 0	Rake	"	
12	8	× 8	5 0	Stanchions	"	
4	12	×12	8 0	Tow Bitts	S. 4 S.	
20	3	×10	9 0	Braces, fore and aft	Rgh.	
72	3	×10	6 0	Braces, thwart ship	"	
62	2½	×12	22 0	Deck Plank	S. 4 S.	C. S.
55	3½	×14	22 0	Bolt Plank	"	"
2	3	×10	22 0	Guards	"	
4	3	×10	32 0	Guards	"	"
4	3	× 8	28 0	Guards	"	"
4	3	× 8	24 0	Guards	"	"
14	3	× 8	22 0	Guards	"	"
1680′	1	×12		Sheeting	S. 2 S.	
120′	1	× 6		Filler	"	
120′	2	× 6		Filler	"	
120′	3	× 6		Filler	"	
10	6	×12	6 0	Rake Braces	Rgh.	
6	3	×12	12 0	Guard	S. 4 S.	

BILL OF IRON

50	$\frac{7}{8}''$ o	31″	Bulkhead, 3, 4	D. B. Clinched
54	$\frac{7}{8}$ o	24	Bulkhead 2	D. B. Plain
60	$\frac{7}{8}$ o	26	Bulkhead 2	D. B. Plain
60	$\frac{3}{4}$ o	21	K. & D. Beams	D. B. Clinched
20	$\frac{3}{4}$ o	11	K. & D. Splices	D. B. Clinched
20	$\frac{3}{4}$ o	24	Transoms	D. B. Plain
40	$\frac{3}{4}$ o	18	Rake Braces	D. B. Clinched
10	$\frac{3}{4}$ o	18	Rake	D. B. Clinched
10	$\frac{3}{4}$ o	24	Rake	D. B. Clinched
48	$\frac{3}{4}$ o	14	Stanchions	Mac. Bolts
48	$\frac{3}{4}$	15½″	Stanchions	Mac. Bolts
16	$\frac{3}{4}$ o	18	Tow Bitts	Mac. Bolts
16	$\frac{3}{4}$ o	28	Tow Bits	Mac. Bolts
70	$\frac{1}{2} \times 12''$		Stanchions	Boat Spikes
900	$\frac{1}{2} \times 6$		Deck Plk	Boat Spikes
950	$\frac{3}{8} \times 7$		Bott Plk	Boat Spikes
500	$\frac{3}{8} \times 7$		Guards	Boat Spikes
300	$\frac{7}{16} \times 8$		Braces	Boat Spikes
½ keg.	20d		Fillers	Galv. Spikes
1 keg.	10d		Sheeting	Galv. Spikes
1700 sq. ft.			P. & B. Paper	
50	1″			Clinch Rings
50	$\frac{15}{16}''$			Clinch Rings
140	$\frac{7}{8}''$			Clinch Rings
140	$\frac{13}{16}''$			Clinch Rings
260	$\frac{3}{4}''$			Cut Washers
6 Gal.				Copper Paint
6 Gal.				P. & B. Paint
1 Bbl.				Pitch
½ Bbl.				Cement
10 Bales				Oakum

Larger scows 80 feet in length, 24 feet beam, and 6 feet 5 inches depth are fully shown in Fig. 283. The longitudinal X bracing is to be added on the sections *B*, or 6-inch solid longitudinal bulkheads substituted. This sized scow carries about 200 yards of riprap rock, or in the neighborhood of 350 tons. Scows of similar construction 30×90×9 feet deep are about as large as can be used in general work. Those constructed for use on river work where there is little liability of rough water are often made much shallower, and of lighter construction. A large scow, 30×90 feet, built by the author, Fig. 284, is shown on blocking in the shipyard and nearly ready for launching.

The costs given in Chapter XI are very low, owing to the proximity to a large supply of cheap timber, and the cost will usually run about 50 per cent. more than the costs given.

The grounding of scows on beaches, and other rough usage, as well as the inroads of the teredo, make it necessary to watch such plant very carefully, and they should be hauled out for general repairs

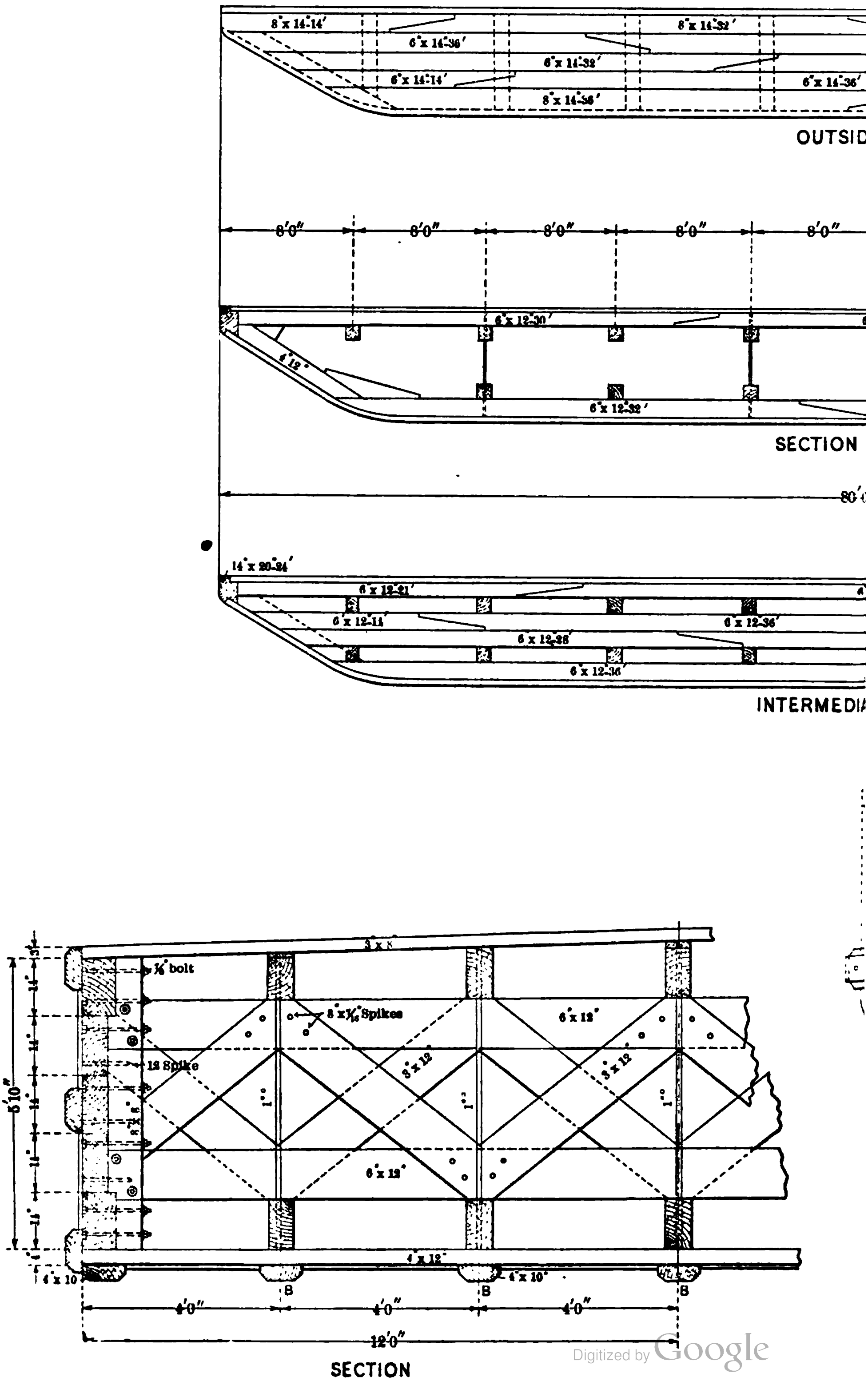
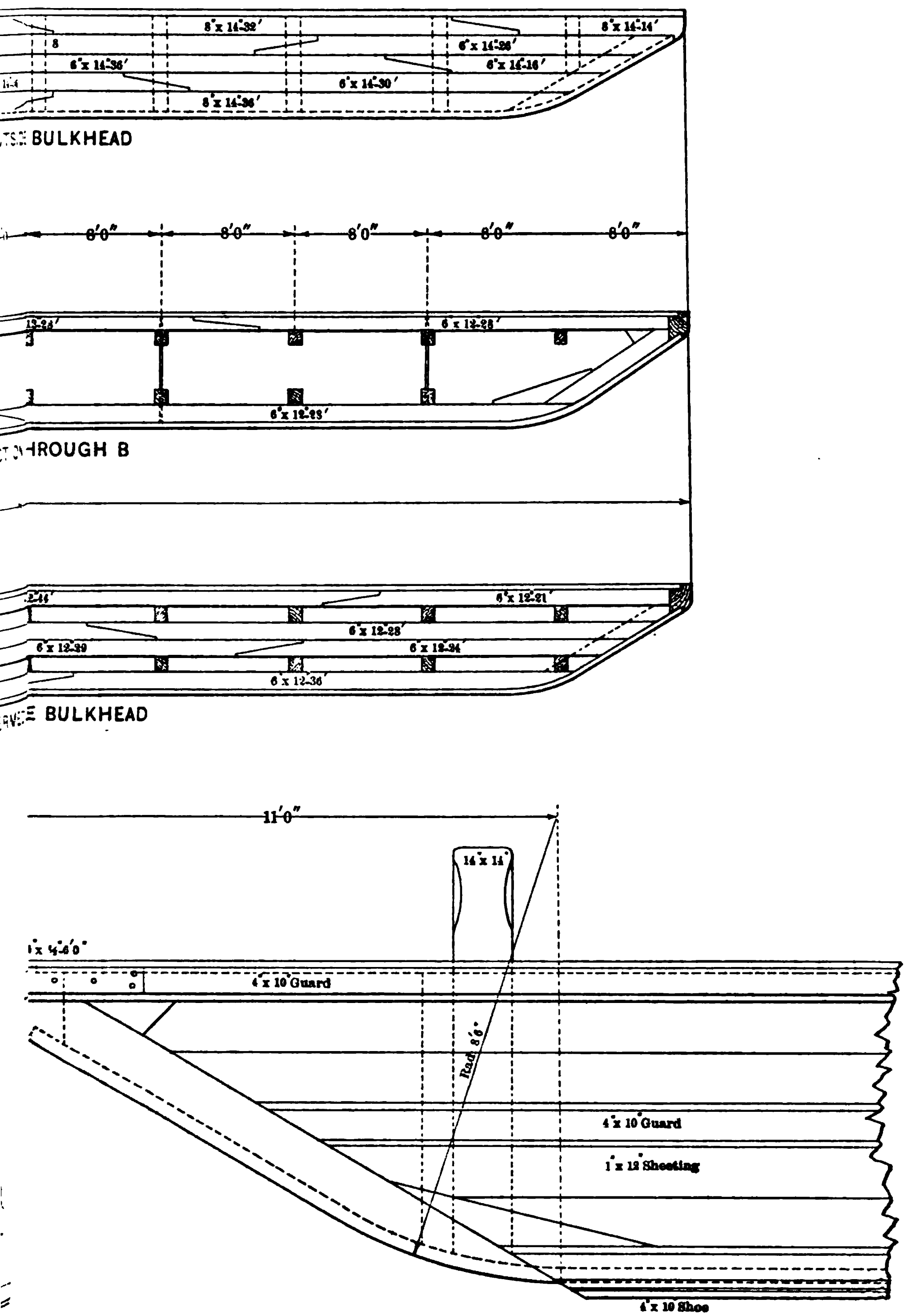


FIG. 283.—Scow or L



...ster, 80 Feet Long.

(To face page 380.)

FIG. 284.—SCOW YARD. DREDGE HULL READY TO LAUNCH.

and copper painting every six months or oftener. Scows can be chartered in most places at about $125 per month for a 22×60 scow, and $150 a month for a 24×80 scow.

Heavy derrick scows can be constructed similar to the clam-shell dredge shown in detail in Fig. 223, or cheaper rigs can be constructed as described and illustrated in Chapter XXIV.

The usual specifications for scows are practically all similar to the following paragraphs, which cover the essential points:

Planking. The sides are to be planked as shown. The planks are to be in long lengths. There are to be no more than three pieces to any one strake in the entire length of the scow, and generally there are to be alternately two and three lengths to each strake. The planks are to be fastened to the stanchions with $\frac{3}{4}$-inch through fastenings clinched over rings, and $\frac{1}{2}$×8-inch ship-spikes, using one through fastening and one spike at each intersection, and at each end of each piece. The guards are to be as shown, and are to be fastened in the same manner, excepting that each end of each piece is to have two through fastenings and the spikes are to be 12 inches long. Each alternate fastening in the top guard timber is to pass through the shelf strake. The bottom planking and guards at the ends are to be similar to the side planking and are to be fastened in the same manner. The deck planking is to be 3×10 inches, as shown, fastened to the beams with $\frac{5}{16}$×6-inch ship-spikes, using two spikes at each end of each plank, and at each intersection with the deck beams. There are to be at least four deck beams between butts in adjoining strakes, and at least four strakes between the butts on a beam. All holes for spikes are to be counter-bored and plugged.

Boring Holes.—The holes for all fastenings are to be bored $\frac{1}{16}$ inch smaller than the iron used, and all drifts are to be cold pointed.

Surfacing.—The keelsons and floor beams are to be surfaced one edge, and all other lumber in the hull is to be surfaced four sides. The dimensions specified or shown on the plans are for the timbers in the rough. A variation from these sizes of $\frac{1}{4}$ inch will be allowed for all planed surfaces.

Calking Seams.—Calking seams are to be provided on the side, bottom and deck planks. They are to be so made that after the pieces are assembled the seams in the side and bottom planks will be $\frac{1}{4}$ inch open and 2 inches deep, and the seams in the deck planks will be $\frac{3}{16}$ inch open and $1\frac{1}{2}$ inch deep.

Calking.—All seams in the side and bottom are to have four threads, and the seams in the deck two threads, of the best oakum,

evenly spun and tucked. The oakum is to be well horsed after each two threads have been put in place and must fill the seams to within $\frac{3}{8}$ inch of the surface of the timber. The seams below water-line are to be cemented and the deck seams are to be payed with pitch and scraped. Above the water-line the side seams are to have two coats of white lead put on with seam brushes.

Fastenings.—All fastenings are to be of the best quality of refined iron or soft steel, and all hull fastenings below the load water-line are to be galvanized. Ship-spikes are to have smooth edges and points. All round fastenings are to be cold-pointed before being driven.

Painting.—The sides and ends of the hull above the middle guard, are to have two coats of paint of an approved brand and color. The bottom and sides below the middle guard are to have two coats of copper paint of an approved make, and applied just before the hull is ready for launching.

Further specifications are given in describing the hull for a clamshell dredge in Chapter XVI, and in Appendix VII for the Government pile-driver hull.

TABLE XXXVI.—GASOLINE TUG ENGINES. DOUBLE CYLINDER.

H.P.	No. of Cyls.	Bore.	Stroke.	R.P.M.	Usual Propeller Diameter. Inches.	Approx. Net Weight. Lbs.	Price F. O. B. San Francisco.	
							Without Whistle Outfit.	With Whistle Outfit.
10	2	6½	6½	400	25	1400	$700.00	$ 735.00
15	2	6½	7½	360	28	2000	875.00	910.00
20	2	7¾	9	330	34	3000	....	1150.00
30	2	9	10½	300	38	4200	....	1550.00
50	2	10½	12	280	46	6250	....	2500.00
65	2	12	14	260	50	8750	....	3000.00

TABLE XXXVII.—GASOLINE TUG ENGINES. FOUR CYLINDER.

H.P.	No. of Cyls.	Bore. Inches.	Stroke. Inches.	R.P.M.	Usual Propeller Diameter. Inches.	Approximate Net Weight. Lbs.	Price F. O. B. San Francisco.
25	4	6	6½	425	28	2100	$1400.00
35	4	6½	7½	375	36	2900	1800.00
50	4	7¾	9	350	40	5000	2500.00
75	4	9	10½	325	46	8000	3400.00
110	4	10½	12	300	52	13000	4750.00
140	4	12	14	280	58	15000	6000.00
175	4	13½	15½	260	60	20000	7000.00
200	4	14½	17	240	66	29000	8000.00

TABLE XXXVIII.—TUGBOAT, SCOTCH MARINE BOILERS.

Shell.		Furnace.		Tubes.		Dome.		Stack.		Square Feet Heating Surface.	Licensed Pressure.	Approximate Weight Complete.
Diameter. Inches.	Length. Inches.	Diameter. Inches.	Square Feet Grate Surface	Number.	Diameter. Inches.	Diameter. Inches.	Height. Inches.	Diameter. Inches.	Length. Feet.			
48	72	22	7¼	62	2	22	16	16	8	180	165	5,000
54	76	26	8¼	58	2¼	26	28	18	8	209	165	7,000
54	84	26	9	58	2¼	26	18	18	8	235	160	7,600
60	84	28	11¼	67	2½	28	20	20	8	294	170	8,400
60	96	28	13	67	2½	28	20	20	8	339	170	9,100
66	96	30	13¾	68	3	30	24	22	12	408	158	11,400
72	96	36	17	75	3	36	30	24	12	454	160	13,500
72	105	36	19	75	3	36	30	24	12	503	160	14,250
78	105	36	19	100	3	36	30	26	15	642	150	17,000
78	120	36	22	100	3	36	30	26	15	747	150	18,500
84	120	40	25	114	3	40	36	28	15	835	155	22,000

TABLE XXXIX.—TUGBOAT, DRY-BACK SCOTCH BOILER.

Shell.		Furnace.		Tubes.		Dome.		Stack.		Square Feet Heating Surface.	Licensed Pressure.	Approximate Weight Complete.
Diameter. Inches.	Length. Inches.	Diameter. Inches.	Square Feet Grate surface.	Number.	Diameter. Inches.	Diameter. Inches.	Height. Inches.	Diameter. Inches.	Length. Feet.			
40	50	18	4.33	48	2	18	12	12	8	90	160	2,800
42	64	20	6.66	50	2	20	16	16	8	132	175	4,000
48	75	22	8.33	70	2	22	18	16	8	202	185	5,500
54	80	26	10.25	73	2¼	26	18	18	8	242	160	7,000
54	88	26	11.50	73	2¼	26	18	18	8	274	165	7,500
60	88	28	12.50	80	2½	28	20	20	8	330	170	9,000
60	100	28	14.6	80	2½	28	20	20	8	387	165	10,000
66	100	30	15	78	3	30	24	22	10	433	158	11,500
72	100	36	18	87	3	36	30	24	12	486	165	13,300
72	109	36	20	87	3	36	30	24	12	542	165	14,200
78	109	36	20	108	3	36	30	26	15	662	150	16,300
78	124	36	23	108	3	36	30	26	15	770	150	17,800
78	132	40	27	94	3	40	36	28	15	760	150	20,000
84	124	40	26	124	3	40	36	28	15	863	150	22,000

Each boiler is fitted with first-class marine trimmings and includes smoke-stack, gates, steam and water gages, firing tools, whistle, pop safety valve and blow-off valve.

CHAPTER XX

THE FOUNDATION

THE coffer-dam is only the means of reaching a desired end, and this must be borne in mind and the construction made as simply as possible to obtain a first-class foundation.

When the coffer-dam is completed and pumped out, work can then proceed if the pumps are able to control the water easily. The character of the foundation having been previously decided upon, after a careful examination of the site, it is assumed that the temporary work has been executed in a manner which is properly related to the permanent structure.

The different kinds of bottom likely to be encountered are: First, light sand and gravel or mud of unknown depth; second, similar material overlying either cemented gravel, clay, hardpan, or rock; third, a clean rock bottom, which is approximately smooth and level; fourth, a sloping rock bottom, which is either smooth or rough, and fifth, a rough and irregular rock bottom.

Should the bottom be of the first kind—light sand and gravel or mud of unknown depth—the soft upper layer may be removed by a dredge previous to the building of the dam, or it may be removed by a dredge or grapple from within the inclosed area, and without the necessity of keeping the dam pumped out, or pumping may be kept up with a dredging-pump and the light material removed in this way, after which the heavier material may be removed as deep as necessary by hand-shoveling and a dirt-box, as shown in Fig. 56. In such a bottom the foundation is usually made by driving piles from 2 to 4 feet centers, this distance being regulated by the bearing power, as determined from Wellington's formula in Chapter IV, and building upon the tops of the piles, after they have been cut off to a level below low water, a grillage of timber. The space between the piles should be filled with broken stone or concrete, and the grillage placed entirely below low water, the coffer-dam being kept pumped out to allow this work to be

done, and also during the laying of the footing courses of the masonry which are below the water.

When the soft bottom overlays good clay, hardpan, or rock, as in the second case, and the depth exceeds 20 or 25 feet below the water-surface, piles may be driven to the harder substratum to act as bearing-piles. But when the depth is in the region of 20 feet or less, it is best to excavate and place the foundation masonry directly upon the solid bottom. The foundation will be of the character described for some of the following cases.

The third class is similar to the foundation at Chattanooga after the gravel was removed. The fissures in the rock are filled up or closed with cement and concrete, and a leveling course of concrete put down on which to found the pier (Fig. 93).

Bottoms of the fourth class should have all the loose and decomposed rock removed and steps cut out by blasting and wedging, to give a secure hold for the foundation, but if it is simply rough and irregular a leveling course of concrete will be all that is required on which to start the pier. Bottoms of clay and hardpan will require a similar treatment, except that the leveling course of concrete must be made of sufficient thickness to properly distribute the pressure, which will seldom be less than 3 feet and can often be increased with advantage to 6 or 8 feet. An example of the stepping of rock bottom was given in the account of the Forth bridge piers in Chapter VIII, and was shown by the dotted lines in Fig. 110.

Where there is a current caused by leakage through the sides of a coffer-dam, or from the bottom, or if the water within the dam is agitated by the pumping, it will be best, after the bottom is clean and properly prepared, to allow the water to run in and then deposit the concrete through the still water. This has been successfully accomplished through 25 or 30 feet of water, and while some engineers recommend allowing the concrete to set from one to three hours before depositing, to prevent the cement from washing out of the concrete, this is not necessary or advisable if the proper care is exercised and the proper apparatus used. The concrete should be made from one-third to one-half richer than would be used for similar open-air work, as there will be some loss of strength.

The simplest method is to deposit the concrete in paper sacks by sliding them down a smooth wooden or iron chute, or by loading them into a box or skip and dumping them out after the box reaches the bottom. The sacks should be of tough paper, similar to flour sacks, and when they reach the bottom they may be broken by a pike-pole and the concrete allowed to run together. Thin

cloth sacks are sometimes used and they become fairly well cemented together by the mortar which oozes through.

Where the amount of concrete is considerable it will be best to use a tube or bottom-dumping box. The use of a steel pipe *trêmie* in the most approved manner is described in Chapter X. For placing concrete under water on the Boucicault bridge over the river Saône in France a wooden tube 16 inches square was used. This is described in the *Engineering News* of May 18, 1893. The tube was carried transversely across the caisson on a traveling-crane which ran lengthwise of the caisson on tracks on the sides, thus allowing the tube to be moved in any desired direction. The tube was built in sections which could be easily removed, was provided with a hopper at the top into which the concrete was dumped, and a drop-door at the bottom to let out the concrete. The tube was filled as it was lowered down into the water, and opened when within 16 inches of the bottom. As concrete was dumped in above, the tube was moved about and a 16-inch layer of concrete deposited. When one layer was complete, another of the same thickness was deposited. This method of using 16-inch layers was said to have obviated laitance or the exuding of the gelatinous fluid which prevents uniform setting. The concrete was deposited about the heads of the piles and no grillage used. The thickness of the concrete, which was deposited at the rate of from 90 to 100 yards per day, was 9.84 feet, and was allowed to set fourteen days before the pier was begun.

A metal tube may be used, such as was employed on the Harvard bridge at Boston by W. H. Ward. This tube (Fig. 285) was not provided with a bottom and the first filling of the tube was consequently done after the tube was lowered and the concrete became somewhat washed. This may easily be prevented by using concrete in paper sacks to fill the tube the first time. The tube was suspended from a derrick and was moved about so as to keep the concrete level and deposit it in layers. This account is taken from Vol. 31 of the *Engineering News*, from which is taken the following description of a metal bucket used by W. D. Taylor on the Coosa River.

This bucket (Fig. 286) was of riveted construction and held one yard of concrete. The maximum depth of water was 26 feet, at which depth the bucket and its load became so lightened that the bucket tripped as soon as the flanges touched the bottom. Similar boxes are often constructed of wood, or they are often made V-shaped, one side being arranged to open and dump the load. The placing of concrete in the dry by a bottom dumping bucket is described in Chapter X.

For concrete work of this character natural cement is often used, but on all important work Portland cement should be employed.

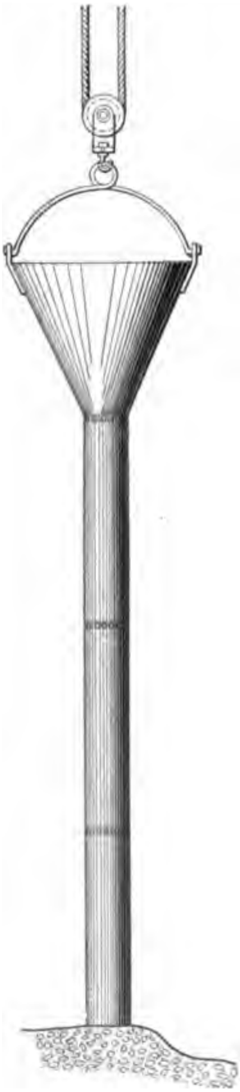


Fig. 285.—Metal Tube for Concreting.

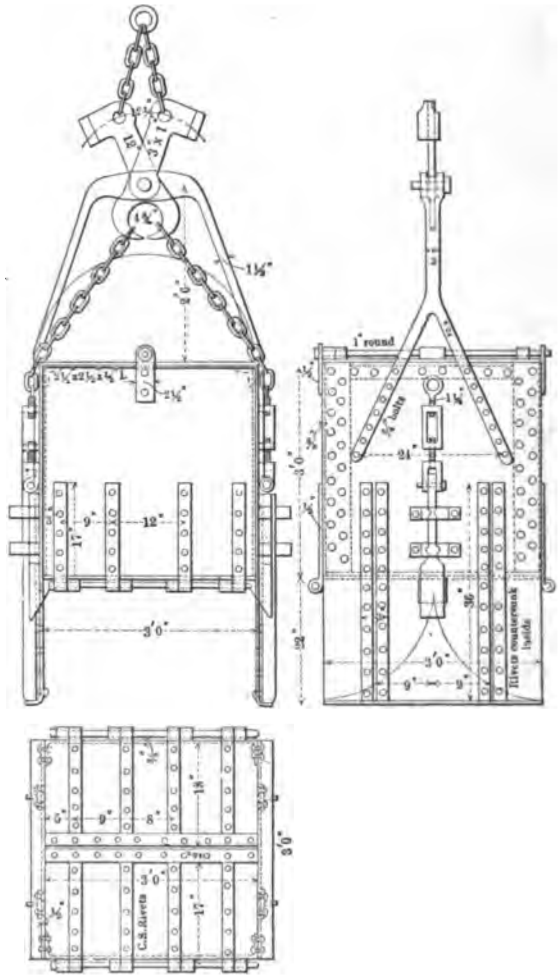


Fig. 286.—Metal Bucket for Concreting.

The proportions range from one of cement, two of sand, and four of broken stone, to one of cement, three of sand, and six of broken stone.

On such a base either a masonry or monolithic concrete pier may be constructed. The pier at Little Rock (Fig. 96) was of this construction and of the composition given in Chapter VII. A similar piece of work was constructed on the Red River bridge on the St. Louis & San Francisco Railway and is described in the *Engineering News* of June 2, 1888, by C. D. Purdon, assistant engineer

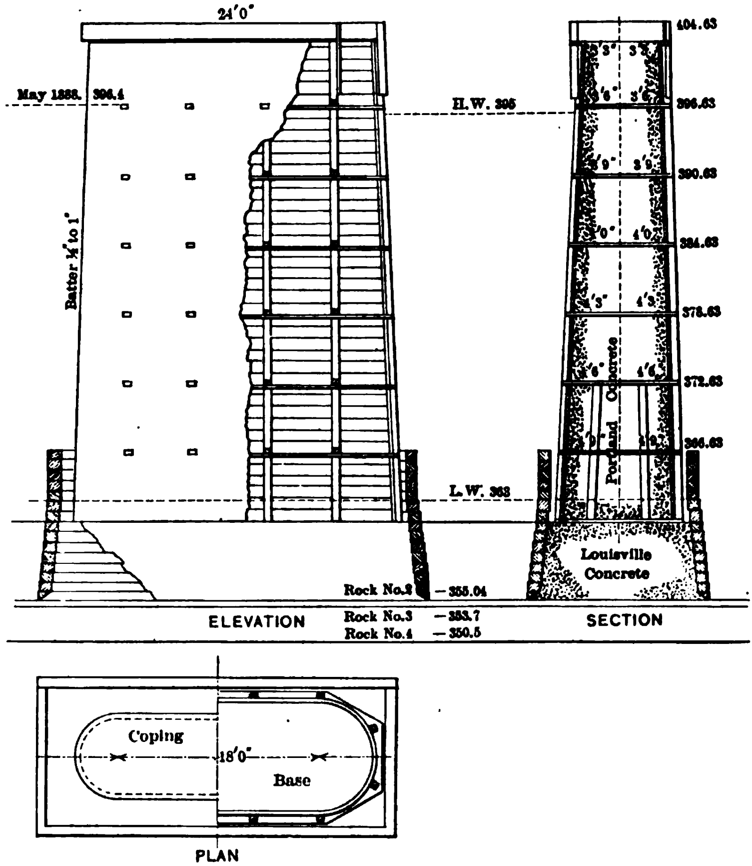


FIG. 287.—CONCRETE PIERS, RED RIVER BRIDGE.

in charge of the work under James Dun, chief engineer. The cribs were filled with Louisville cement concrete up to within 2 feet of low water, on which was built the pier. (Figs. 287 and 288.)

"After the crib had been filled with concrete and the surface leveled off, the center lines of the pier were located and a frame of 2×8-inch plank, the shape of the pier, and 4 inches larger to allow

for lagging, was placed in exact position and held by pieces spiked to the crib. On this frame upright posts 6×6 inches and 5 feet 10 inches high, with a batter of ½ inch per 3 feet, were set in the position shown on the drawing, then the feet spiked to the frame and another frame similar to the first, but 6 inches narrower, placed on them. This again was brought to exact position and braced to the crib and the frame completed by putting lagging of 2-inch

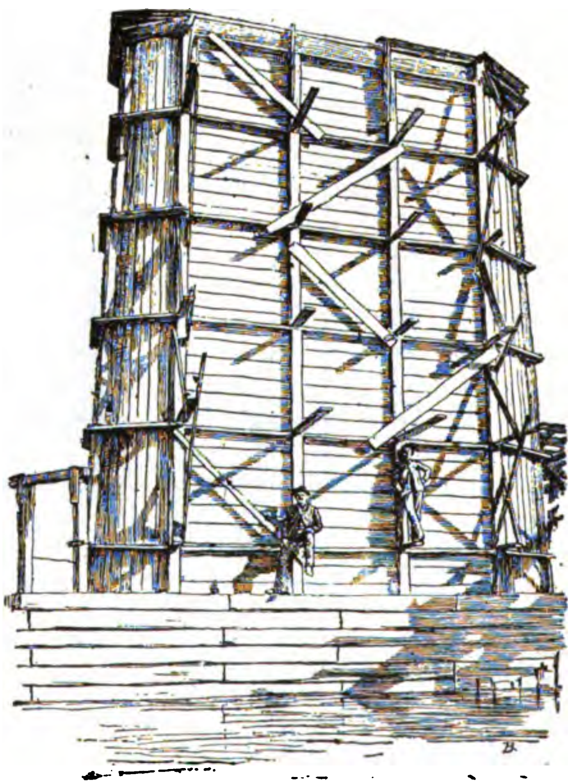


Fig. 288.—Concrete Forms, Red River Bridge.

plank inside the posts and spiking to them. This lagging was horizontal in the body of the pier and vertical (2×4 inches) at the ends, beveled pieces being introduced in the ends at intervals to make up the difference of the upper and lower circles. Next 2×6-inch planks were placed across on the tops of the posts, running clear through the pier, to act as braces. In the rest of the frames these braces were allowed to extend about 6 feet on each side and

the frame braced by spiking plank to them and to the vertical posts. After a section of frames was completed a bed of cement mortar about 2 inches thick was spread all over the concrete in the crib. On this rough stone, in such pieces as one man could easily handle, was placed so that no two pieces would be closer than 2 inches, nor any piece within 2 inches of the frame, the stone being thoroughly wet before laying.

"Next, on this course of stone another bed of mortar was placed, sufficient to fill all the spaces between the stones and remain about 2 inches thick above them. It was then well rammed with rammers made by inserting a handle in a section of a pile, except at the edges, where a rammer made of a 2-inch plank cut in the shape of a spade was used, to insure a perfect skin of cement without any breaks. After this had been well rammed, another layer of stone was placed and covered with mortar as before, and so on.

"The coping, which was made similar to the body of the pier, was finished by about 1½ inches of cement mixed with sand one to one, fluid enough to be struck off by a straightedge, the top of the frame being dressed and leveled for that purpose.

"After the pier had been completed the frames were removed and the braces running through the piers cut off by a chisel inside the concrete. Then, to make a smooth surface, the pier was thoroughly wet and plastered with a mixture of 1 part sand to 1 part cement, after all the rough or loose portions had been scraped off. This was mainly done for appearance."

The mortar for the body of the pier was made of 1 part Alsen's German Portland cement and 4 parts of sand. There was used about 1⅓ barrels of cement to a cubic yard of completed pier. In mixing the mortar eleven ordinary pails full of water were used to one barrel of cement, which caused the water to just appear on the surface when the tamping was done.

The lock walls on the Illinois and Mississippi canal have been constructed of monolithic concrete under Captain W. L. Marshall, Corps of Engineers. The work was executed under L. L. Wheeler, engineer in charge, from whose account, in the report of the Chief of Engineers for 1894, the following is taken:

"The rules adopted for the work were adhered to and are worthy of careful study.

"I. The forms or molds of the walls will be divided by vertical partitions perpendicular to the longest axis of the mass, and the walls be constructed by filling alternate sections.

"II. The sections will be filled in horizontal layers, well rammed,

each layer to be deposited before the ‘initial set’ of the previously deposited layer. When the work of filling a section is begun it must proceed without intermission to completion, working night and day if necessary.

“III. The facing and backing must go on simultaneously in the same horizontal layers, using the same cement in the facing as in the backing, with no defined lines of demarcation between the facing which contains no stone and the concrete backing.

“IV. When the top surface of the coping is reached it will be finished after ramming by cutting off the excess by a straightedge, and rubbed smooth and hard by a float. No plastering or wet finishing will be allowed.

“V. The facing of the walls will not be finished by plastering or washing with cement after the forms are removed, nor dressed in any manner beyond chiseling away rough ridges, should the plank forming not be smooth.

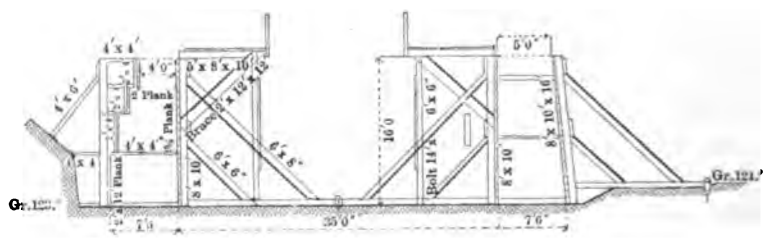


FIG. 289.—CONCRETE FORMS, ILLINOIS AND MICHIGAN CANAL.

“VI. The concrete shall be mixed with all the water it will take without water showing after ramming, or without ‘quaking’ upon ramming.

“VII. At such intervals as may be necessary vertical wells, at least 1 foot square, will be formed along the middle of the masses of concrete, reaching to near the bottom thereof. The masses of concrete after forming will be kept sheltered from the sun, the outer surfaces kept moist and the wells kept filled with water until well set, or about three weeks. The walls will then be filled with concrete.

“VIII. In preparing the cement for mixing with other ingredients of concrete, from five to ten barrels will be kept thoroughly mixed dry, to guard against chance barrels of defective cement, and the necessary quantity of cement will be taken for each batch from this mixture.

“IX. Two cements of different qualities shall not be used in the same section, but as far as practicable each mass shall be homo-

geneous throughout, but a slight excess of cement in the facing to reduce its capacity to absorb water."

The rate at which the concrete was deposited in the work was determined by the rate of ramming, and but one yard every five minutes was deposited. The forms (Fig. 289) were lined with dressed pine plank 4×8 inches on the face, of uniform thickness, and with 2-inch rough plank on the back.

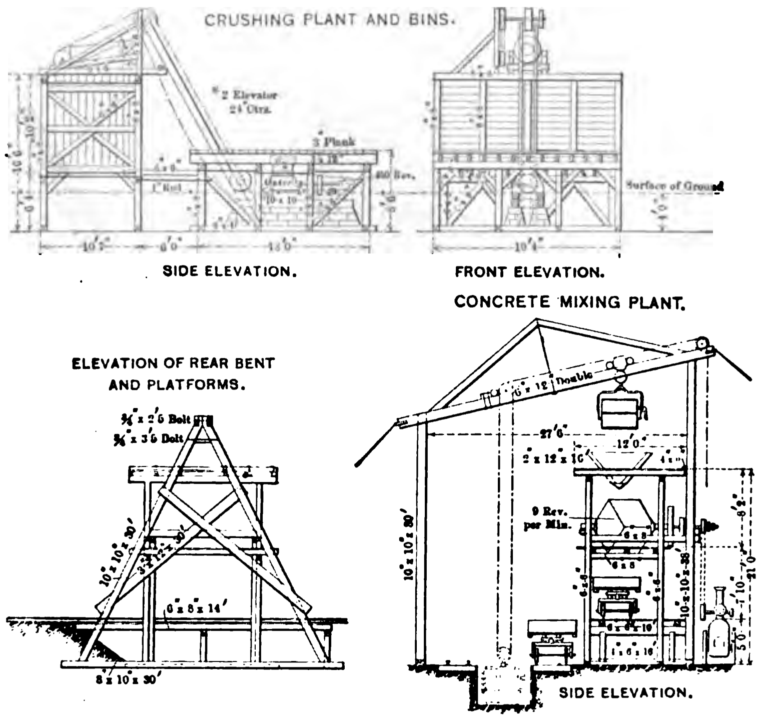


FIG. 290.—STONE CRUSHER AND CONCRETE MIXER, ILLINOIS AND MICHIGAN CANAL.

Rough plank is sometimes used on such work and lined with oiled paper, or ordinary dressed plank may be used and coated with soft soap. In most sections of the country crushed broken stone can be obtained, but owing to the magnitude of this work a crusher was built (Fig. 290) and was found to work very satisfactorily. The concrete mixer shown in Fig. 290 was operated by a 15-horse-power portable engine. The proportions finally adopted for the concrete were 1 of cement, $3\frac{1}{3}$ of gravel, and 4 of broken stone, while the facing and coping were composed of 1 part cement and 2 parts of clean river sand.

That the sand for concrete be clean and sharp is very essential, and any loam or dirt must be washed out. Equally essential is good, clean, sharp, broken stone without dust or dirt. The cement used on the above work was a German Portland, but most of the American Portlands are first class and will give as good results as the imported.

Where good, fresh, cement is being supplied a few tests to a car-load will be sufficient, and for cements of the grade of Atlas or

FIG. 291.—REINFORCING BAR BENDER.

Empire the guarantee of the manufacturer, supplemented by a few tests, should be sufficient. But for cements which have been shipped by water, tests should be made from every five or ten barrels.

The Atlas Cement Company recommend, for concrete laid in open air on moist ground where great weight must be carried, 1 of cement, 2 of clean sharp sand, and 4 of 2-inch broken stone; this sand and cement to be thoroughly mixed dry, then just enough water added to thoroughly moisten, and the mass turned over at

least twice, when the stone is to be added in a thoroughly wet condition. This must then be put at once into the molds and well rammed.

Where a solid bottom is to be built upon, the proportions of 1 of cement, 3 of sand, and 6 of broken stone are recommended. For ordinary construction 1 of cement, 4 of sand, and 8 of broken stone, while to obtain a concrete as strong as ordinary natural

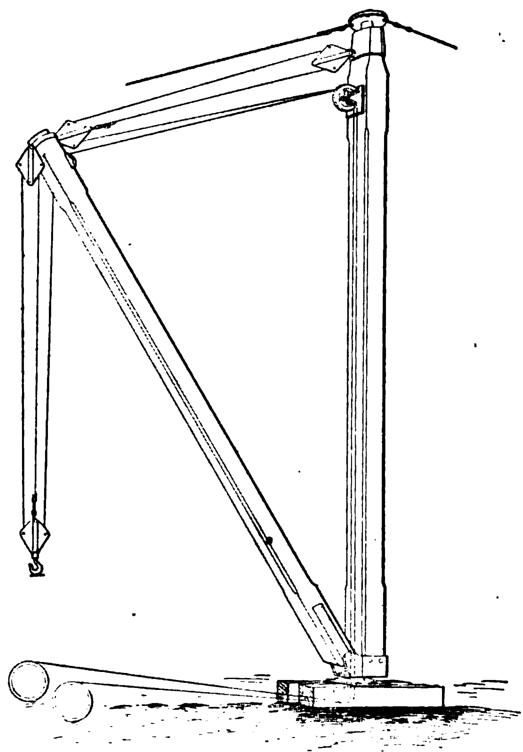


FIG. 292.—DOUBLE-DRUM GUY DERRICK, AMERICAN HOIST AND DERRICK CO.

cement concrete, 1 of cement, 5 of sand, and 10 of broken stone can be used.

The average cost in the western portion of the United States of such concretes, including labor, tools, timber forms, and a fair profit to the contractor, would be for the first $8 per cubic yard, for the second $7.50, for the third $7.25, and for the fourth $7. In the Eastern States they can be figured at $1 per cubic yard less.

Where reinforcing is to be used in foundation concrete it is very often necessary to bend the reinforcing bars. A tool for this purpose,

such as is shown in Fig. 291, will bend up to 1¼-inch high-carbon bars without heating. It can be operated by one man, by the ratchet lever, and the dies can be adjusted for the thickness of the steel to be bent by means of the set-screws. The machine weighs about 350 pounds, and costs $100.

Where the leveling course of concrete has been put in and the pier is to be of stone, the footing course should be of carefully selected material. They should be large stones with good beds, and should be as thick or preferably thicker than the courses above. Where the bearing pressure does not exceed two tons per square foot, the footing courses may be stepped by allowing each course

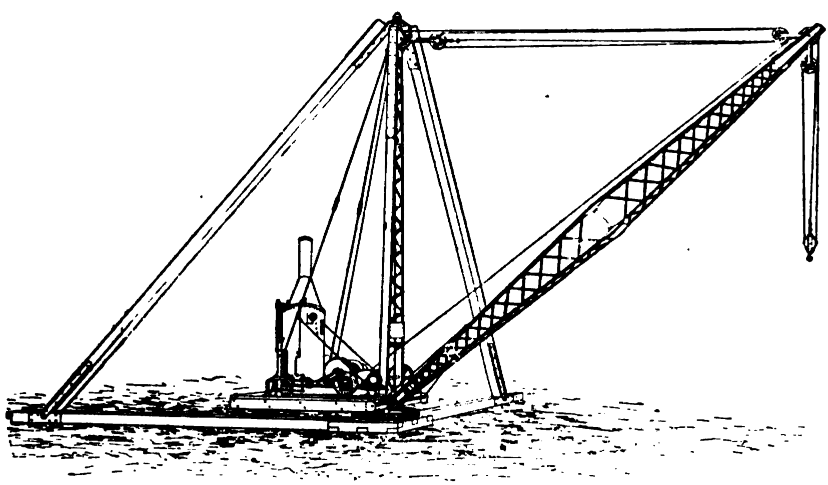
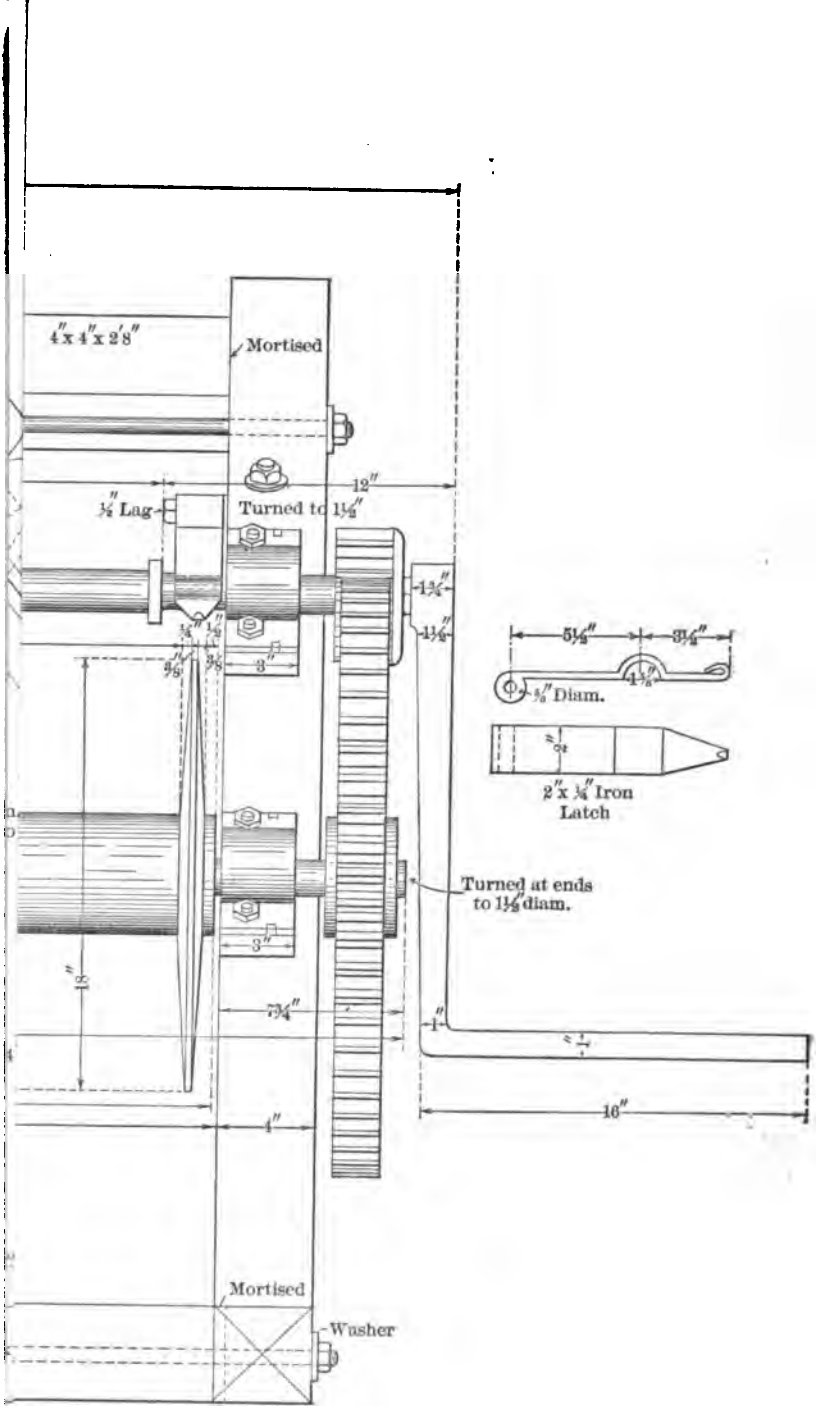


Fig. 293.—Stiff Leg Derrick with Steel Boom.

to project about one and one-third times its thickness, depending of course on the quality of the stone.

The usual way of handling the material for foundations and piers is to boat it to the site, where it is placed by a stiff-leg derrick, or, if guys can be used, by a derrick with wire-rope guys.

The stiff-leg derrick shown in Fig. 293 is fitted with steel mast and boom, but where timber is plenty the entire rig can be made with large timbers similar to the stiff-leg derrick on the scows, shown in Chapter XXIV, and the author has used fir booms in one stick up to 90 feet long, as shown in Fig. 190. Everything about a stiff-leg derrick must be carefully proportioned, including the goose necks and all of the connection plates and pins.



(To face page 396.)

Derrick lines and other hoisting lines can be handled by crabs or winches. A style of winch used by the author for many years is shown in Fig. 294 in full detail, and can be easily constructed by ordinary first-class workmen.

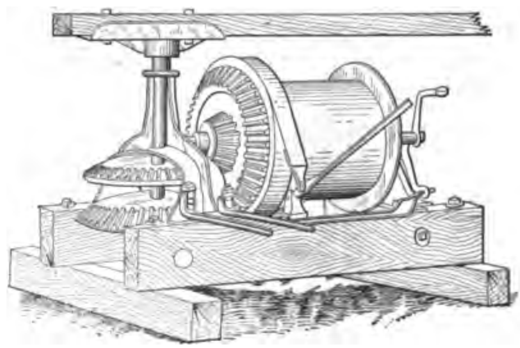


FIG. 295.—SINGLE-DRUM HORSE-POWER, CONTRACTORS' PLANT MFG. CO.

The fittings for such derricks can be obtained from a number of firms, an American Hoist and Derrick Company outfit being shown in Fig. 292. This is rigged to be operated by a double-drum hoist, which can be one operated by horse-power (Fig. 295) if the piers are near the bank and if steam-power is not available. The usual form, however, is a double-drum steam-hoist like the Lidgerwood machine shown in Fig. 296.

Instead of swinging a derrick by hand lines or by tag lines running through snatch blocks to the nigger heads, it is best to employ slewing engines similar to the one shown in Fig. 297, which can be operated by the hoisting engineer, and save one or two men on the work. This kind of a rig used in connection with a bull-wheel on the derrick will pay for itself usually in two or three months' time. Hoist engines fitted with two reversing drums for swinging the boom are all similar to the engine shown in Fig. 298. This style of an engine will work well on land, but not so well on a scow, which is apt to list and cause an especially hard pull. The sizes and data of hoist engines are shown in Tables XL and XLI.

FIG. 296.—DOUBLE-DRUM HOIST-ENGINE, LIDGERWOOD MFG. CO.

Vertical boilers are often required about a contractor's plant similar to the one illustrated in Fig. 299, and are fully described in Table XLII. All boilers should be built to comply with the most

rigid requirements of City Boiler Inspection Ordinances, and one somewhat exceeding these requirements, built under Government Navy specifications, is shown in Fig. 300. This boiler, however, is of such a size that it should not be used for small construction, owing to the expense of moving and setting it up, and boilers of

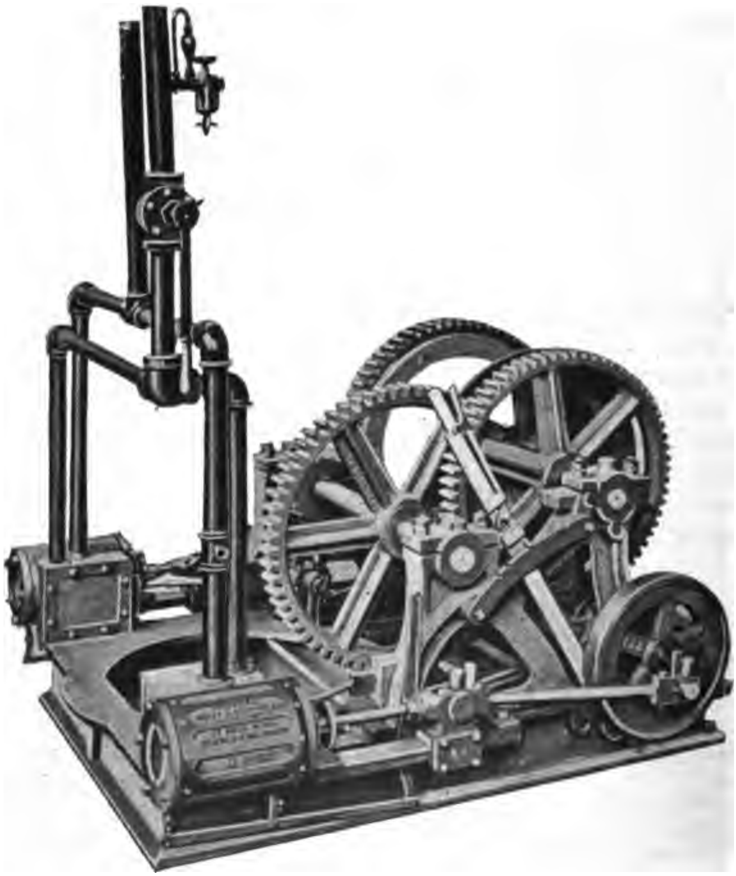


FIG. 297.—SLEWING ENGINE FOR DERRICK.

the locomotive type described in Chapter V, are much more easily transported and placed.

Where electric power is available an electric hoist (Fig. 301) should be used, as it will be found much more convenient.

Works of any magnitude should, however, be fitted from the beginning with a cableway, which will avoid the necessity of boat-

ing the materials, erecting of large derricks, and facilitate in every way the prosecution of the work, besides often making a balance on the right side of the ledger. The Lidgerwood cableway on dam No. 11 of the Great Kanawha River, a tower of which can be seen

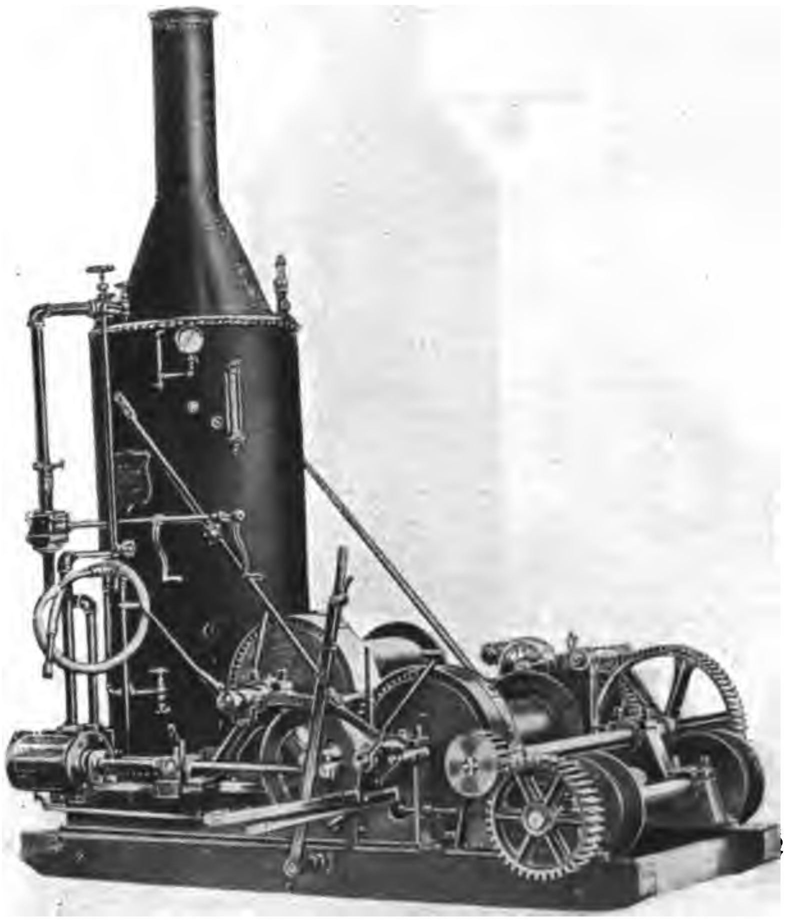


FIG. 298—DERRICK ENGINE WITH TWO REVERSING DRUMS FOR SLEWING BOOM.

in Fig. 12, had a span of 1505.5 feet and carried a net load of four tons on a main cable $2\frac{1}{2}$ inches in diameter. The stone quarry was located on one bank, and the stone was taken directly to the stone-yard and to the work in the river. A seam of coal in the quarry also

supplied fuel for the dredges and pumps, the coal being handled by the cableway, as was also the material from the railroad siding on the opposite bank.

The details of these cableways have been developed and perfected to a wonderful extent, as a result of their use on the Chicago drainage

FIG. 299.—VERTICAL BOILER.

channel. The engine for operating one of these with a capacity of eight tons has double 10×12-inch cylinders, the cranks being set at an angle of 90°, and is provided with reversing link motion. The double drums regulate both the hoist at a speed of 300 feet per minute and the travel along the cable at 1000 feet per minute. A 70-horse-power boiler is required.

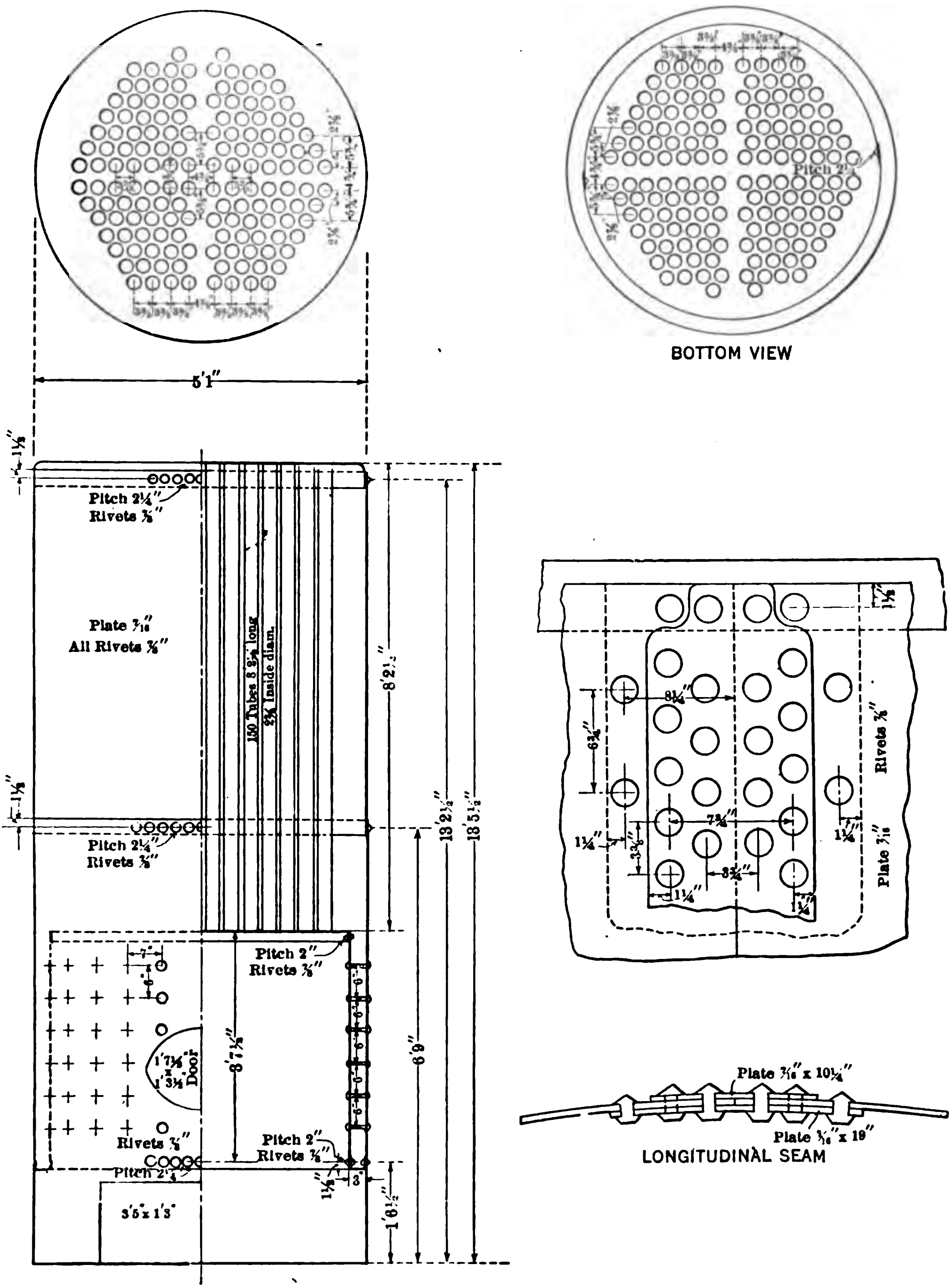


FIG. 300.—VERTICAL BOILER, NAVY TYPE.

The carriage and skip, which are automatic in action, are shown in Fig. 302, the capacity of those on the drainage channel being 1.8 yards, and the average of a month being about 600 yards per day of ten hours. The cost of operation, including labor, fuel, and everything except interest on plant and repairs, was less than $18 per day or from 3 to 4 cents per yard.

The cableway on the Coosa dam and lock (Fig. 303) had a capacity of about eight tons and made a round trip on an average of about three minutes. Such a plant is out of reach of high water and of trains where used over railroad tracks as at the North Avenue bridge in Baltimore.

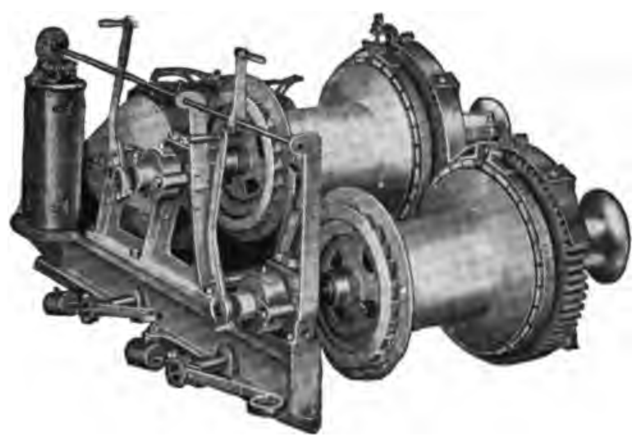


FIG. 301.—LIDGERWOOD ELECTRIC HOIST.

The Court Street stone-arch bridge at Rochester, N. Y., of eight spans, was constructed with the aid of a cableway, which was also used to remove the old bridge and piers. A cableway of one span was used to construct the Melan concrete-arch bridge at Topeka, Kan. The bridge has five spans and a total length of 650 feet. During the extreme high water in the early part of 1897, when everything was completely inundated, and an ordinary derrick plant would have been swept away, the cableway was high and dry out of reach of the flood.

The prevailing low prices of contract work make it necessary to employ every improvement on important engineering work, and the

cableway has doubtless come to stay as one of the most remarkable of our tools.

Very often it becomes necessary to use jacks of large capacity around foundation work, and while the hydraulic jack is very satisfactory when it is in good shape and in warm weather, the Norton jack, Fig. 304, can be purchased up to 100 tons capacity, and will usually be found much more satisfactory. This is a ball-bearing ratchet-screw jack, and worn or broken parts can be easily replaced.

They are useful around caisson work for jacking-up during the

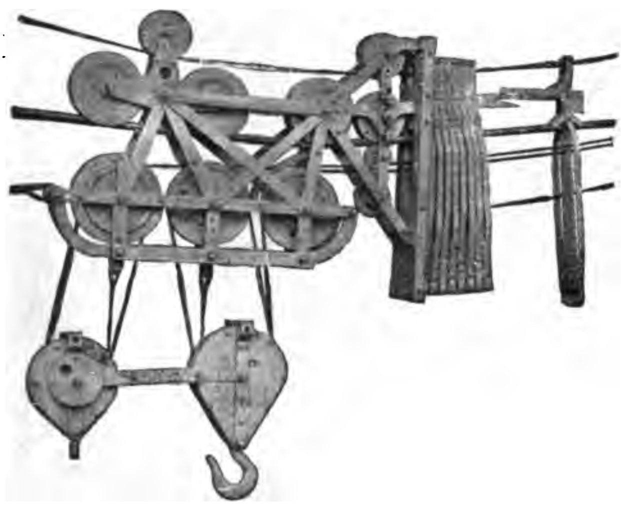


FIG. 302.—LIDGERWOOD HIGH-SPEED CABLEWAY CARRIAGE AND FALL ROPE CARRIERS.

building of the cribs and in launching them when they must be started with one or more jacks.

After caissons are in place, it is often necessary to use jacks to loosen up braces, so they can be removed or replaced. The same work in cofferdams can be done more easily by the use of some form of jack, and, if others are not at hand, ordinary screw jacks may be employed.

The necessity for the proper lighting of foundation work where night crews are employed is one of the most important matters to be considered by the engineer, and wherever electric current can be obtained electric lights should be used instead of gasoline torches,

which are very unsatisfactory, especially in windy weather. A direct-connected DeLaval turbine generator, as shown in Fig. 305, occupies

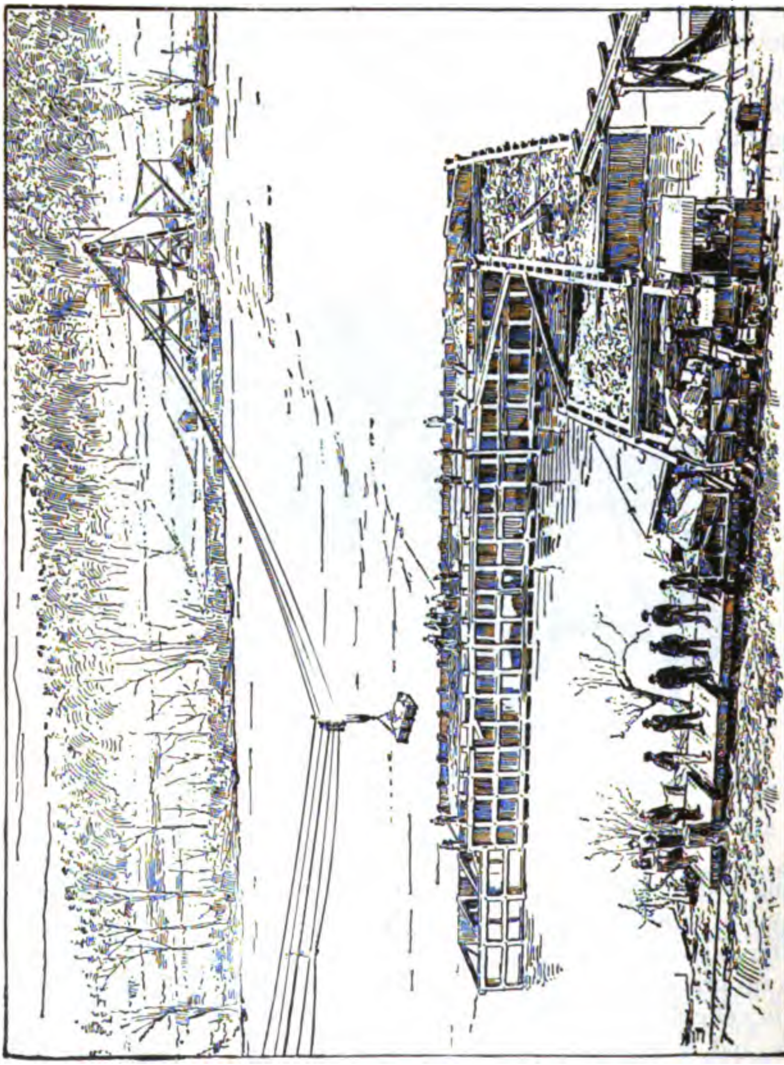


FIG. 303.—LIDGERWOOD CABLEWAY AT COOSA DAM. SPAN 1012 FEET.

but little space and can be used to advantage on any work where steam boilers are in use.

Very often it is more convenient to use an acetylene light as shown in Fig. 306, which is a Milburn portable lamp, and which can be

FIG. 304.—NORTON BALL-BEARING RATCHET SCREW JACK.

FIG. 305.—DE LAVAL TURBINE DRIVING GENERATOR.

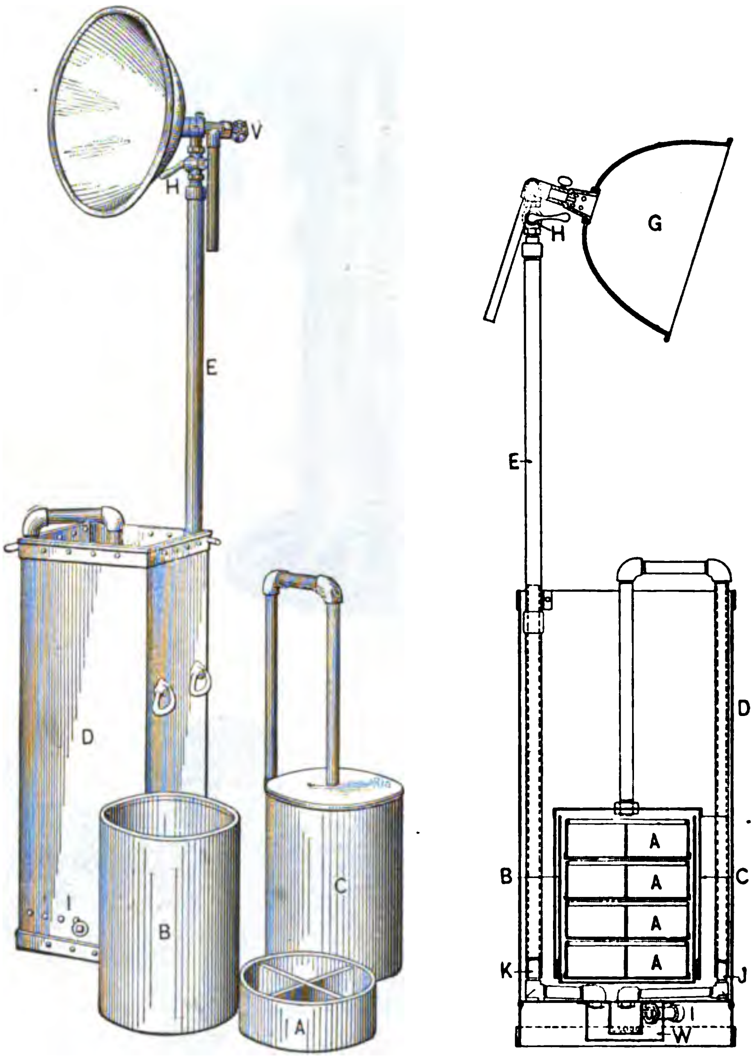


FIG. 306.—MILBURN PORTABLE LIGHT.

obtained up to 12,000 c.p. Usually one of about 1500 c.p. is all that will be required for ordinary work, and should more light be needed additional ones may be added as required.

TABLE XL.—TABLE OF SIZES, LIDGERWOOD SINGLE-CYLINDER, SINGLE-DRUM HOISTING-ENGINES.

Horse-power Usually Rated.	Dimensions of Cylinder.		Weight Hoisted Single Rope, Usual Speed. Pounds.	Suitable Weight of Pile-driving Hammer for Quick Work. Lbs.	Dimensions of Hoisting-drum.			Dimensions of Bed-plate		Dimensions of Boiler.			Estimated Shipping Weight Complete, etc. Lbs.
	Diameter. Inches.	Stroke. Inches.			Diam. Body between Flanges, Inches.	Length Body between Flanges. Inches.	Diameter Flanges. Inches.	Width. Inches.	Length. Inches.	Diameter Shell. Inches.	Height Shell, Inches.	Number of 2-inch Tubes.	
4	5	8	1200	1000	10	20	22	38	60	28	63	40	3550
6	6¼	8	1500	1250	10	20	22	38	60	28	69	40	3950
8	6¼	10	1750	1500	12	20	24	41	73	30	72	44	4850
10	7	10	2500	1800	12	20	24	41	73	32	75	48	5050
11	7	10	2500	2000	14	22	26	45	73	34	78	52	5350
12½	8¼	10	4000	2500	14	23	29	47	73	36	75	57	6550
15	8¼	10	4000	2800	14	23	29	47	73	36	81	57	6750
20	8½	12	6000	4000	16	26	33	54	84	40	84	80	8500
25	10	12	8000	5000	16	26	33	54	84	42	90	88	9500

TABLE XLI.—TABLE OF SIZES, LIDGERWOOD DOUBLE-CYLINDER, DOUBLE-DRUM HOISTING-ENGINES.

Horse-power Usually Rated.	Dimensions of Cylinders.		Dimensions of Hoisting-drums.		Weight Hoisted Single Rope, Average Speed.	Suitable Weight of Pile-driving Hammer for Quick Work.	Dimensions of Boiler.			Dimensions of Bed-plate.		Estimated Shipping Weight with Boiler Complete.
	Diameter. Inches.	Stroke. Inches.	Diameter. Inches.	Length. Inches.			Diameter. Inches.	Height. Inches.	Number of 2-in. Tubes.	Width. Inches.	Length. Inches.	
8	5	8	12	22	2,000	1,500	32	75	48	47	80	6,500
12	6¼	8	14	22	3,000	2,000	36	75	57	50	86	8,000
16	6¼	10	14	26	4,000	2,800	38	81	68	54	89	9,000
20	7	10	14	26	5,000	3,500	40	84	80	54	89	9,550
30	8¼	10	14	27	8,000	5,000	42	90	88	57	94	11,400
40	8½	12	16	32	10,000	8,000	50	102	124	70	117	21,000
50	10	12	16	32	12,000	10,000	53	102	150	70	117	22,000

TABLE XLII.—VERTICAL BOILERS.

Nominal Rated Horse-Power.	Shell.			Fire Box.		Mean Thickness.		Size of Steam Outlet.	Tubes.				Total Heating Surface.	Stack.		Shipping Weight Pounds Approximate.		
	Diam. In.	Height over all. Ft.	Height over all. In.	Diam. In.	Height. In.	Shell. In.	Heads. In.	In.	No.	Diam. In.	Length. Ft.	Length. In.	Sq. Ft.	Diam. In.	Length. Ft.	Boiler Only.	Fixtures Only.	Boiler and Fixtures.
6	28	5	10	$23\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{4}$	42	2	2	8	75	10	15	1209	391	1600
8	30	6	5	$25\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{2}$	48	2	3	3	99	12	15	1446	454	1900
10	30	7	4	$25\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{2}$	48	2	4	2	122	12	20	1593	507	2100
15	34	7	8	$29\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	2	70	2	4	6	186	14	25	2058	642	2700
20	36	8	9	$31\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	2	76	2	5	7	245	15	25	2645	755	3400
25	38	9	3	$33\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$2\frac{1}{2}$	88	2	6	1	304	16	30	3153	947	4100

Each boiler, on cast-iron base plate, is furnished with grate bars, safety valve, steam gage and siphon, glass water gage and gage cocks, check valve, stop valve, blow-off cock, smokestack, hoods and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

CHAPTER XXI

THE FOUNDATION (CONTINUED)*

THE determination of the exact character of the foundation is entirely dependent upon the bearing capacity of the soil or foundation bed. In cases where there is large doubt as to what the bottom will carry per square foot of surface, it is always best to make some tests to arrive at definite conclusions; but in ordinary cases it is possible to adopt figures well within safe limits, and thus avoid the trouble and expense of experimenting. Unless the experiments are very carefully conducted, precedent is the better method.

For the State Capitol at Albany, N. Y., one of the most important structures in the United States, very careful and elaborate experiments were conducted by W. J. McAlpine, the engineer in charge of the work; and as the material, which was blue clay, was found to sustain a load of 6 tons per square foot, it was decided to adopt 2 tons as the safe load to put upon the foundation bed.

The Congressional Library at Washington, D. C. (Fig. 307), another very important building, had the foundation constructed to come within $2\frac{1}{2}$ tons per square foot, although the yellow clay was found to carry a total load of $13\frac{1}{2}$ tons per square foot.

It is always possible, of course, to thoroughly drain the foundations of a building, or to at least know the exact condition in which the foundation bed will exist; but for bridge work the circumstances are very different, and after the foundation is once in place, unless it be on solid rock, examinations are very difficult, so that it is necessary to be much more sure as to what the material will carry.

The Bismarck bridge (Fig. 308) across the Missouri River, on the line of the Northern Pacific Railway, has the piers (Fig. 309) founded upon the clay, which was found to sustain a load of 15 tons per square foot before settlement ensued, and the actual load is 3 tons per square foot. From a report on this work the following account is taken of the character of the foundation bed:

* See formula for bearing on clay, Antwerp Quay Wall, Chapter XXIX. Also Chapter XXV.

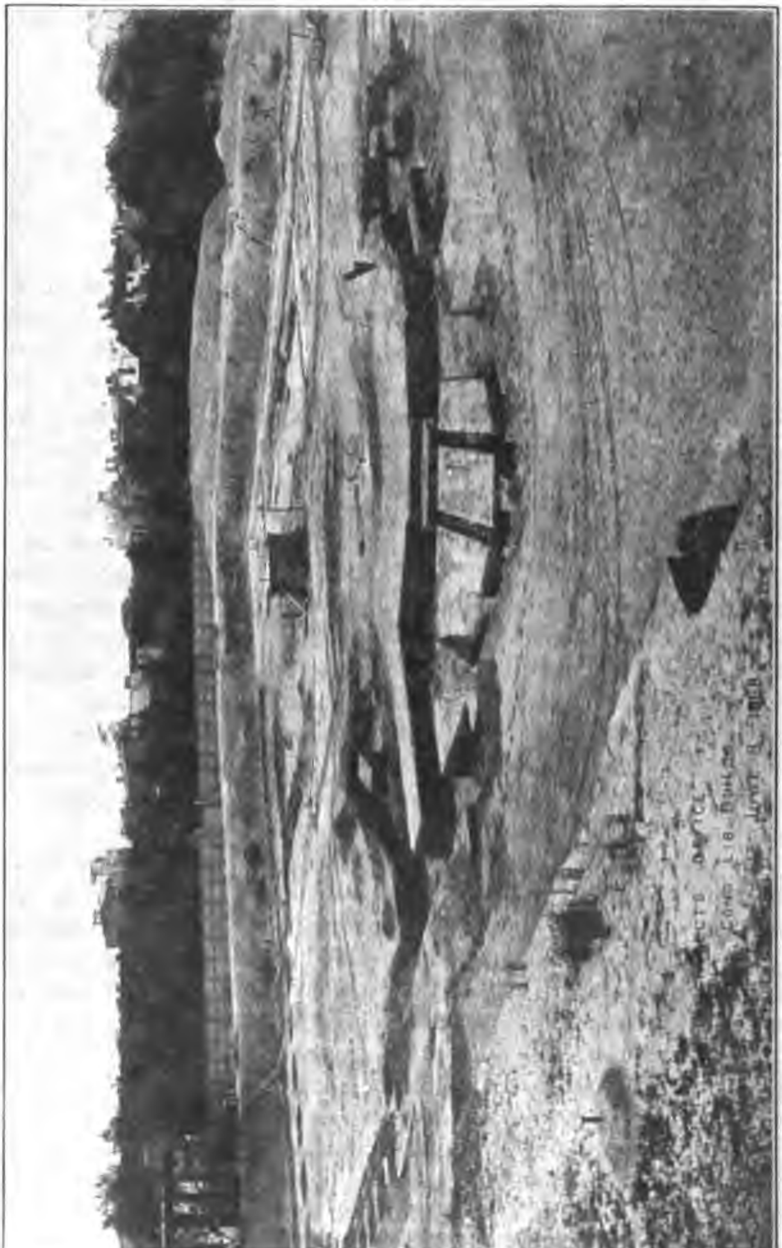


FIG. 307.—CONGRESSIONAL LIBRARY, WASHINGTON, D. C.

Fig. 308.—Bismarck Bridge, Northern Pacific Railway.

"With the exception of some thin strata of soft sandstone of irregular thickness and extent, no rock is found in position in this part of Dakota. The entire country is underlaid with a very hard stratified clay, the depth of which has not been ascertained. Borings proved this clay to be at least 100 feet thick on the line of the bridge, and a hole intended for an artesian well has since been sunk within the Bismarck city limits to a depth of over 1300 feet in the clay. This clay, however, is in many respects more like a rock than a clay; small specimens tested for compression have sustained a weight of over 300 pounds per square inch without crushing, and when they gave way yielded like rock, and showed no tendency to bulge out at the sides. Water has little or no effect upon this clay, even where the current is extremely strong, but when exposed to the dry air the clay slakes rapidly and crumbles to pieces."

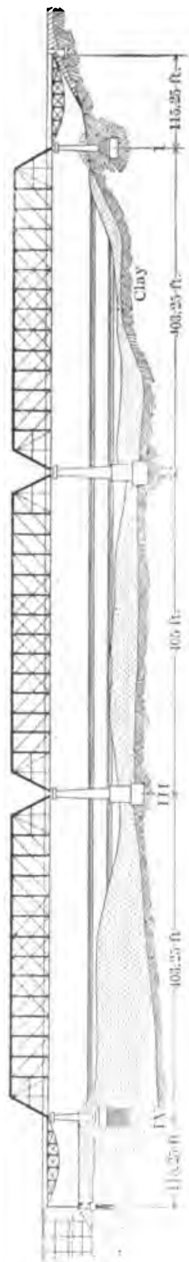


Fig. 309.—Bismarck Bridge Foundations.

The Roebling Suspension bridge at Cincinnati, Ohio, has the towers (Fig. 310) founded on coarse gravel, although by going 12 feet deeper solid limestone could have been reached. The load of 3.63 tons per square foot adopted was, however, well within safe limits for so large a spread as the base of the towers.

"No definite plan of foundation had been fixed upon; none could be safely adopted before the excavations had been sunk some considerable depth. Whether a solid rock bottom could be reached on either side was uncertain; so also was the question of piling or solid layers of timber left open. It is known that the bed of the Ohio River is throughout its extent underlaid with rock, at no great distance below the surface. But the depths at which the rock is found varies very considerably. At all the rapids and shallows the rock forms the bed itself; in the pools between, heavy layers of gravel, sand, or mud are deposited. At or near the mouths of tributaries, the rock has generally been excavated by the action of the water

to a great depth; the soft material has taken its place, and consequently good foundations can only be made by heavy expenditures. At the site of the bridge, a bed of blue limestone and shale, not very solid, underlies the river bed at a depth of about 12 feet below the lowest part of the channel. A short distance above the bridge this shale is laid bare at low water on the Covington side, a part of it extending

FIG. 310.—CINCINNATI SUSPENSION BRIDGE.

half way across. Under the Covington tower, a heavy bed of coarse sand, mixed with gravel, is found above the rock, while the surface layer is composed of the original clay bottom which forms the river banks. On the Cincinnati side, the original clay or loamy bottom has, to some extent, been washed away, and latterly been filled up again by the materials obtained from cellar excavations.

"In this artificial bank the excavation of the Cincinnati tower was commenced about the 1st of September, 1856, and sunk down to the level of the river, which, during this and the next two months, fell to low-water mark. A little rise of 4 feet intervened, but the river fell again, and continued low until the month of December. It was owing to this remarkably favorable state of the river that we so well succeeded in our foundations, and at a cost which must be considered as very small, considering their magnitude and the sudden floods which may occur at almost any time and sweep over and destroy costly preparations.

"By a wise resolution of the Board of Managers, the work was not to be commenced before a bona fide cash subscription of $300,000 had been secured. Contrary to my expectations, this subscription was rapidly obtained; and in view of the promising state of the river, it was concluded to forthwith commence the foundation work. But no preparations whatever had been made, no materials on hand, no machinery, and no efficient pumps. The total want of the latter proved a very serious drawback, and seriously threatened to defeat the enterprise at the very outset. It is true, the city was full of steamboat-pumps, but of such small dimensions and such construction that they were of no account in such an operation. Raising clean water is an easy process, but to raise large masses of soft mud and sand is not so easy. After experimenting and losing a few precious weeks in an endeavor to work some patent rotary pumps, which utterly failed, we came very near to a complete halt. There was no time to get proper steam-pumps of large dimensions constructed at any of the shops in Cincinnati, nor could we expect them in time from the East; every day's loss was irreparable—and so we were thrown back upon our own resources. Accordingly, I had four large square box-pumps constructed of 3-inch pine plank, strongly hooped, and 24 feet long; one pair 18 inches in the clear, the other 20 inches. Cast-iron gratings with large india-rubber flap-valves formed the piston and lower check-valves; heavy piston-rods, connected with chains which passed over sheaves, and were shackled to rods, extending over the coffer-dam down to the river. These pumps were put up vertically, in two pairs, one pair worked at a time. They were propelled by one of the engines of the then *Champion No. 1*, a powerful towboat, owned by Amos Shinkle, Esq., who generously placed it at my disposal. These pumps worked well and never failed; they threw mud and sand as effectively as pure water, and discharged 40 gallons at each lift.

"When the Cincinnati excavation was commenced, a strong

oak sheet-piling was driven along the river-front to guard against the pressure of water. This, together with a solid embankment, proved a most efficient coffer-dam on the river side. Owing to its low stage, the river gave us no trouble at all. But by the great depth and extent of the foundation, most of the wells along the rising ground, back of Cincinnati, were laid dry. We drained their supplies, and had to pump them out; and this copious influx came from a quarter totally unexpected. The excavation, however, proceeded rapidly day and night, until all the clay and sand was removed, and a deep layer of coarse sand and gravel was laid bare. Soundings were now made by driving long iron bars to the limestone shale underneath, which proved to be about 12 feet lower. A depth of over 12 feet below extreme low water was reached, and the question now arose, whether to go to the rock, to pile, or to lay down a solid timber platform.

"A compact bed of gravel, if left undisturbed, and protected against undermining and washing, stands next to a solid rock foundation, provided that unequal settling is guarded against. Had this tower been located inside of low-water mark, I should have decided upon going down to the rock, although one season's loss would have been the consequence. Piling I considered inferior to the plan adopted, to say nothing of the loss of time. It was therefore decided to stop at the gravel, and to build a solid timber foundation up to low-water mark, thence to commence the masonry. If the timber could be obtained in time in sufficient quantity, the success of this kind of foundation was much more certain to be achieved, and with less risk and cost, than any other plan.

"The timber foundation, thus laid, forms a platform of 110 feet long by 75 feet wide, composed of twelve courses or layers next to the river, but stepped off towards the land side to eight courses, in consequence of the greater hardness of the gravel, almost equal to hardpan. We were obliged to employ all kinds of timber, soft and hard, mixed, as white pine, oak, maple, hickory, button-wood, elm, beech. The length of logs also varied from 25 to 40 feet. They were all flattened and counter-hewed to an even thickness of 12 inches, leaving the other two sides rough. The courses were crossed at right angles; each stick was thoroughly secured by iron rag-bolts 18 inches long and 1 inch in diameter. The vertical joints were left open and filled with clean gravel and broken stone. Every course was leveled off with the adze and then thoroughly grouted with cement grout before the next course was laid down. Care was also taken in breaking the longitudinal joints efficiently. A solid platform of

timber, 110 feet long, 75 feet wide, 12 feet deep on the river side, and 8 feet on the land side, well put together in the manner described, offers a foundation nearly as good as rock, provided it is guarded against undermining.

"The result has fully justified my expectation. I have not been able to discover any settlement during the progress of the masonry; its condition to-day proves the excellence of the foundation. There are 16,000 perches of 25 cubic feet, equal to 400,000 cubic feet, of solid masonry in each tower. Allowing 150 pounds as the average weight of one cubic foot, the total weight of one tower is 60 millions of pounds, or 30,000 tons net. The area of the timber foundation being $110 \times 75 = 8250$ superficial feet, the weight upon each foot is 3.63 tons or 7272 pounds, or $50\frac{1}{2}$ pounds per superficial inch. This is equivalent to a solid mass of iron of 15 feet depth. Now experience proves that such a weight of iron will be supported upon a clay floor, if its surface is well consolidated by tamping. In the case of high chimney-stacks, elevated 300 to 400 feet, a still greater pressure is sometimes produced upon each superficial foot."

The great Brooklyn bridge, also constructed under John A. Roebling, had the towers landed on a few feet of sand overlying bed-rock, and the load was allowed to run up to $5\frac{1}{2}$ tons per square foot.

The bridges in London, England, are nearly all founded upon the stratum known as "London Clay," and the Charing Cross bridge causes a pressure upon it of in the neighborhood of 9 tons per square foot, while on the Cannon Street bridge the load runs to in the neighborhood of $6\frac{1}{2}$ tons per square foot. Both of these structures, however, have shown considerable settlement, and when the great Tower bridge was designed it was decided to reduce these loadings very materially. Tests made by sinking a trial cylinder showed settlement under $6\frac{1}{2}$ tons per square foot, and, disregarding skin friction of the caisson and the buoyancy of the water, 4 tons was adopted as the safe load; although, taking these into account, the actual load per square foot was between 1 and 2 tons.

The cantilever bridge at Memphis, Tenn. (Fig. 311), constructed by George S. Morison, has piers founded upon pneumatic caissons, and tests were made to determine the maximum bearing capacity of the soil, which was a compact clay, and it was found to have an ultimate bearing capacity of $9\frac{1}{2}$ tons.

Although the foregoing represent what is the best practice in regard to the allowable loads per square foot on various classes of soil, they have in many cases been much exceeded. For example,

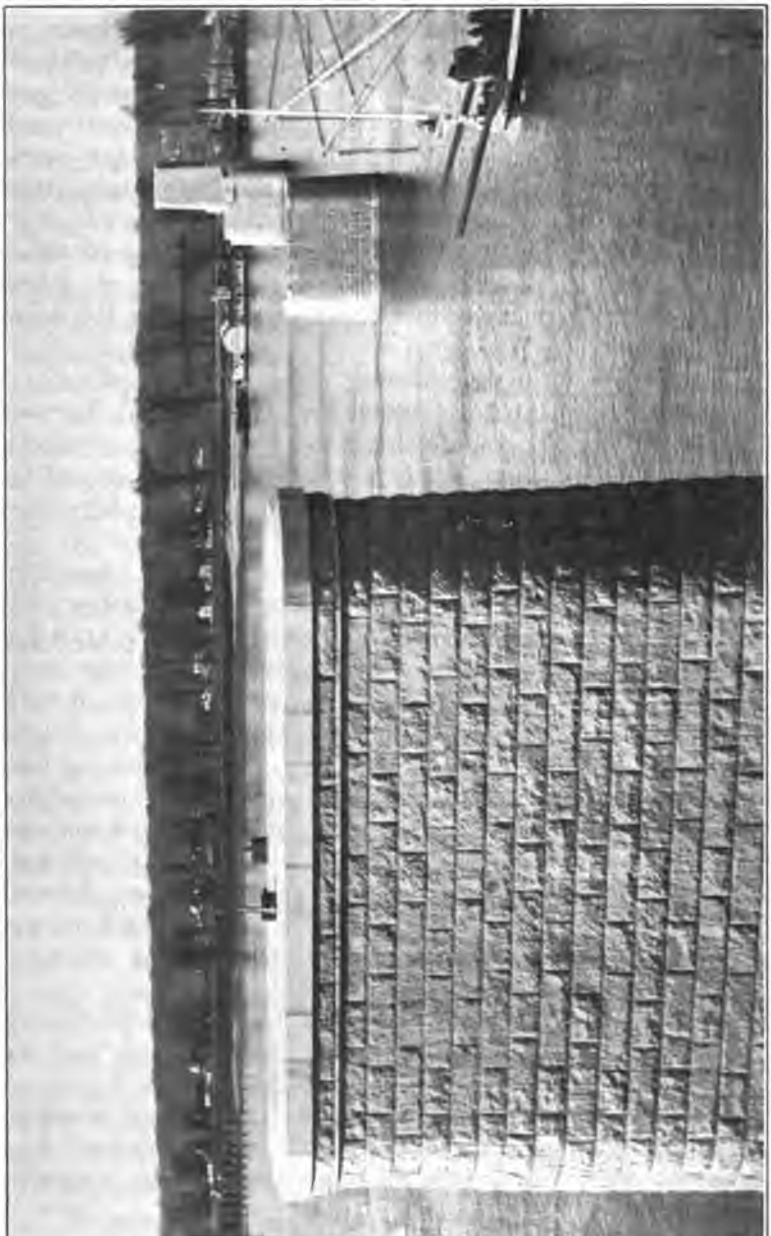


FIG. 311.—PIERS OF MEMPHIS CANTILEVER.

the foundation of the Washington Monument causes a pressure on the very fine sand of 11 tons per square foot, although a maximum of about 14 tons is reached during a high wind. Similar overloaded conditions exist with many bridges, as the Gorai bridge causes a pressure of 9 tons on the close-sand foundation, and nearly as much pressure is put upon the sand bottom in the Nantes bridge, where pressure reaches $8\frac{1}{2}$ tons per square foot, although this has settled some, indicating that such high figures are nearer the ultimate bearing capacity than safe loads. A mixture of clay and sand on which the Szegedin bridge in Hungary is founded carries a load of $7\frac{1}{2}$ tons per square foot, although it was found necessary to relieve this foundation by driving piles.

The above data as to the carrying capacity of soil of various kinds may be supplemented by stating that the safe loads for soils per square foot may be rapidly increased for hardpan, cemented gravel, and, of course, very largely increased for rocky ground, as in the case of the Roquefavour aqueduct in France, where then pressure reaches 15 tons per square foot.

Probably the most generally accepted values for foundation loads, that is, the amount which can be placed with safety upon a square foot of foundation bed, are those given by Prof. I. O. Baker in an article published in the *American Architect*, in which he states the maximum allowable load to be 25 tons per square foot for rock of similar hardness as is used in the best ashlar masonry, 15 tons per square foot for rock equal to the best brick masonry, 5 tons per square foot for rock equal to poor brick masonry, 4 tons for dry clay, 2 tons for moderately dry clay, 1 ton for soft clay, 8 tons for cemented gravel and coarse sand, 4 tons for compact and well-cemented sand, 2 tons for clean, dry sand, and 0.5 ton for quicksand and alluvial soils, although these figures can be increased from 25 to 100 per cent., depending upon circumstances and as the judgment of the engineer on the work may dictate.

The building laws of Greater New York are more generally used as a model and authority than any other building rules, and the following extracts cover the provisions as to foundations:

"Where no test of the sustaining power of the soil is made, different soils, excluding mud, at the bottom of the footings, shall be deemed to safely sustain the following loads to the superficial foot, namely:

"Soft clay, 1 ton per square foot;

"Ordinary clay and sand together, in layers, wet and springy, 2 tons per square foot;

"Loam, clay, or fine sand, firm and dry, 3 tons per square foot;

"Very firm, coarse sand, stiff gravel, or hard clay, 4 tons per square foot, or as otherwise determined by the Commissioner of Buildings having jurisdiction.

"Where a test is made of the sustaining power of the soil the Commissioner of Buildings shall be notified so that he may be present in person or by representative. The record of the test shall be filed in the Department of Buildings.

"When a doubt arises as to the safe sustaining power of the earth upon which a building is to be erected the Department of Buildings may order borings to be made, or direct the sustaining power of the soil to be tested by and at the expense of the owner of the proposed building.

"The loads exerting pressure under the footings of foundations in buildings more than three stories in height are to be computed as follows:

"For warehouses and factories they are to be the full dead load and the full live load established by this code, which also gives the loads from other buildings.

"Footings shall be so designed that the loads will be as nearly uniform as possible and not in excess of the safe bearing capacity of the soil, as hereinbefore given.

"Every building, except buildings erected upon solid rock or buildings erected upon wharves and piers on the water-front, shall have foundations of brick, stone, iron, steel, or concrete laid not less than 4 feet below the surface of the earth, on the solid ground or level surface of work, or upon piles or ranging timbers when solid earth or rock is not found.

"Piles intended to sustain a wall, pier, or post shall be spaced not more than 36 nor less than 20 inches on centers, and they shall be driven to a solid bearing if practicable to do so, and the number of such piles shall be sufficient to support the superstructure proposed.

"No pile shall be used of less dimensions than 5 inches at the small end and 10 inches at the butt for short piles, or piles 20 feet or less in length, and 20 inches at the butt for long piles, or piles more than 20 feet in length.

"No pile shall be weighted with a load exceeding 40,000 pounds.

"When a pile is not driven to refusal, its safe sustaining power shall be determined by the following formula: Twice the weight of the hammer in tons multiplied by the height of the fall in feet divided by least penetration of pile under the last blow in inches

plus one. The Commissioner of Buildings shall be notified of the time when such test piles will be driven, that he may be present in person or by representative.

"The tops of all piles shall be cut off below the lowest water-line.

"When required, concrete shall be rammed down in the interspaces between the heads of the piles to a depth and thickness of not less than 12 inches and for 1 foot in width outside of the piles.

"Where ranging and capping timbers are laid on piles for foundations, they shall be of hard wood not less than 6 inches thick and properly joined together, and their tops laid below the lowest water-line.

"Where metal is incorporated in or forms part of a foundation it shall be thoroughly protected from rust by paint, asphaltum, concrete, or by such materials and in such manner as may be approved by the Commissioner of Buildings.

"When footings of iron or steel for columns are placed below the water-level, they shall be similarly coated, or inclosed in concrete, for preservation against rust.

"When foundations are carried down through earth by piers of stone, brick, or concrete in caissons, the loads on same shall be not more than—

"Fifteen tons to the square foot when carried down to rock;

"Ten tons to the square foot when carried down to firm gravel or hard clay;

"Eight tons to the square foot in open caissons or sheet-pile trenches when carried down to rock.

"Wood piles may be used for the foundations under frame buildings built over the water or on salt-meadow land, in which case the piles may project above the water a sufficient height to raise the building above high tide, and the building may be placed directly thereon without other foundation.

"Foundation walls shall be construed to include all walls and piers built below the curb level, or nearest tier of beams to the curb, to serve as supports for walls, piers, columns, girders, posts, or beams.

"Foundation walls shall be built of stone, brick, Portland cement concrete, iron, or steel.

"If built of rubble stone or Portland cement concrete, they shall be at least 8 inches thicker than the wall next above them to a depth of 12 feet below the curb level; and for every additional 10 feet, or part thereof, deeper they shall be increased 4 inches in thickness.

"If built of brick, they shall be at least 4 inches thicker than the wall next above them to a depth of 12 feet below the curb level;

and for every additional 10 feet, or part thereof, deeper they shall be increased 4 inches in thickness.

"The footing or base course shall be of stone or concrete, or both, or of concrete and stepped-up brickwork, of sufficient thickness and area to safely bear the weight to be imposed thereon.

"If the footing or base course be of concrete, the concrete shall not be less than 12 inches.

"If of stone, the stones shall not be less than 2×3 feet, and at least 8 inches in thickness for walls; and not less than 10 inches in thickness if under piers, columns, or posts.

"The footing or base course, whether formed of concrete or stone, shall be at least 12 inches wider than the bottom width of walls, and at least 12 inches wider on all sides than the bottom width of said piers, columns, or posts.

"If the superimposed load is such as to cause undue transverse strain on a footing projecting 12 inches, the thickness of such footing is to be increased so as to carry the load with safety.

"For small structures and for small piers sustaining light loads, the Commissioner of Buildings having jurisdiction may, in his discretion, allow a reduction in the thickness and projection for footings or base courses herein specified.

"All base stones shall be well bedded and laid crosswise, edge to edge.

"If stepped-up footings of brick are used, in place of stone, above the concrete, the offsets, if laid in single courses, shall each not exceed 1½ inches, or if laid in double courses, then each shall not exceed 3 inches, offsetting the first course of brickwork, back one-half the thickness of the concrete base, so as to properly distribute the load to be imposed thereon.

"If, in place of a continuous foundation wall, isolated piers are to be built to support the superstructure, where the nature of the ground and the character of the building make it necessary, in the opinion of the Commissioner of Buildings having jurisdiction, inverted arches resting on a proper bed of concrete, both designed to transmit with safety the superimposed loads, shall be turned between the piers. The thrust of the outer piers shall be taken up by suitable wrought-iron or steel rods and plates.

"Grillage beams of wrought iron or steel resting on a proper concrete bed may be used. Such beams must be provided with separators and bolts inclosed and filled solid between with concrete, and of such sizes and so arranged as to transmit with safety the superimposed loads.

"All stone walls 24 inches or less in thickness shall have at least one header extending through the wall in every 3 feet in height from the bottom of the wall, and in every 3 feet in length, and if over 24 inches in thickness, shall have one header for every 6 superficial feet on both sides of the wall, laid on top of each other to bond together, and running into the wall at least 2 feet.

"All headers shall be at least 12 inches in width and 8 inches in thickness and consist of good flat stones.

"No stone shall be laid in such walls in any other position than on its natural bed.

"No stone shall be used that does not bond or extend into the wall at least 6 inches.

"Stones shall be firmly bedded in cement mortar and all spaces and joints thoroughly filled."

The subject of Allowable Pressure on Deep Foundations is discussed in a valuable monograph by the eminent engineer, Elmer L. Corthell. As Consulting Engineer for the port works of the Argentine Government, it became necessary to arrive at a reliable figure for the pressure proper to allow on the bottom for sea-walls to be founded by compressed air. Finding the data to be very unsatisfactory, Mr. Corthell caused a search to be made, and has published in the monograph mentioned the results of his investigations. As every engineer engaged in foundation work should own a copy of this book, only the conclusions drawn by Mr. Corthell will be given:

"This analysis is based on the various classes of material so far as they could be ascertained and classified.

"The pressures of stable structures on fine sand range from 2.25 tons of 2000 pounds to 5.80 tons, with an average of 4.5 tons with ten examples.

"On coarse sand and gravel from 2.40 tons to 7.75 tons, with an average of 5.1 tons with thirty-three examples.

"On sand and clay from 2.5 tons to 8.5 tons, with an average of 4.9 tons with ten examples.

"On alluvium and silt from 1.5 to 6.2 tons, with an average of 2.9 tons with seven examples.

"On hard clay from 2.0 tons to 8.0 tons, with an average of 5.08 tons with sixteen examples.

"On hardpan from 3.0 tons to 12.0 tons, with an average of 8.7 tons with five examples.

"The above cases show no settlement. The range is considerable, and, no doubt, in the case of the minimum pressure a much larger weight could have been imposed on the material without producing

settlement. So that, for a safe rule, the average is low and a safe one would lie somewhere between the averages above given and the maximum pressures.

"We find three cases where notable settlement took place in fine sand where the range was from 1.8 ton to 7.0 tons, and the average was 5.2 tons; no doubt the case of the minimum was one of loose quicksand unconstrained.

"In clay—largely cases of London clay—we find five examples where the pressures range from 4.50 tons to 5.60 tons, with an average of 5.2 tons, quite uniform pressures.

"In silt and alluvium we have two cases of settlement which were 1.6 ton and 7.6 tons, a wide variation.

"There are three cases of failure on sand and clay mixed, the pressures ranging from 1.6 tons (Chicago) to 7.4 tons, an average of 3.3 tons. It is to be noted that there was given above an average of 4.9 tons, and ten examples, ranging from 2.5 tons to 8.5 tons, where no settlement ocurred in similar material.

"The records of frictional resistance are quite variable also. In ten cases of cylinder piers, the average was 540 pounds per square foot, ranging from 300 to 1500 pounds, gravel appearing to show the greatest amount (1500 pounds), and mud the least.

"In respect to masonry piers, of which we have twenty-three examples, the range is from 300 pounds per square foot in sand and gravel to 1000 pounds in sand and clay, with an average of 522 pounds. Walls, quays and otherwise, show an average of 270 pounds per square foot, with a range from 205 pounds to 450 pounds with five examples."

Some very valuable data on the bearing capacity of clay is given in Chapter XXIX on the Antwerp quay wall, and a very valuable formula given for varying depths. A general formula will be found in Chapter XXV.

CHAPTER XXII

LOCATION AND DESIGN OF PIERS

PIERS of a bridge cannot always be located with reference to easy construction nor spaced at economical distances apart. In thickly settled parts of a country, or as part of an existing line of communication, the bridge must be located usually in a position previously determined, and the piers can only be spaced with regard to economy, provided due regard can at the same time be paid to the needs of navigation, government requirements, and sufficient waterway.

Where the bridge is to be constructed in a new country, or upon a new line of road, the crossing should be selected where the river is of moderate width; that is, not so wide as to demand a structure of excessive length and probably of excessive cost, nor so narrow that the current will be exceedingly swift and make the foundations very difficult and costly to build, unless, of course, it is narrow enough to admit of using a one-span structure at a reasonable cost. The two-hinged plate-girder arch, Fig. 312, constructed by the author at Youngstown, Ohio, has a span of 210 feet, and while constructed largely for æsthetic reasons, it was also economical on account of the very considerable height, to avoid a pier in the river.

On all the large navigable rivers, the channel is fixed and the length of the channel span prescribed by law, as is also the method of procedure in obtaining the approval of the government engineers. The Secretary of War must be furnished with a copy of the State law authorizing the construction of the bridge, certified to by the Secretary of State under seal; drawings in triplicate showing the general plan of the bridge; a map in triplicate showing the location of the bridge, giving, for the distance of one mile above and one-half mile below the proposed location, the high- and low-water lines upon the banks of the stream, the direction and strength of the current at high and low water, with the soundings accurately showing the bed of the stream, and the location of any other bridge or bridges, such map to be sufficiently in detail to enable the Secretary

FIG. 312.—PLATE GIRDER ARCH, YOUNGSTOWN, OHIO.

of War to judge of the proper location of the bridge. In addition to the above, if the applicant is a corporation, there will be required a certified copy of its articles of incorporation, a certified copy of the minutes of the organization of the company, and an abstract of the minutes of the corporation, showing the present officers of the company, all duly certified to.

When the location of the bridge has been made, a thorough examination of the site must be instituted. Soundings must be made to determine the depth of the stream at low water. Ordinary and extreme high-water lines must be established and the flow of the stream be obtained. A careful examination must be carried out as to the character of the river bed, and drillings made to learn the character and thickness of strata and the distance to bed-rock, as well as the quality of it.

Borings to a small depth may be made by hand-drills (Fig. 313, *A*), which are operated by striking with a sledge and turned constantly

A

B

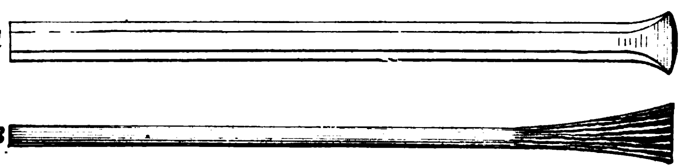


FIG. 313.—HAND-DRILL AND SWAB.

to keep a round bore, or if long and heavy they will cut their way, if simply raised up and allowed to drop, with their own weight. The hole is kept partly filled with water and can be cleaned out with a small sand-pump or with a swab (Fig. 313, *B*) made from a stick slivered at the end, which will also bring up samples.

The Pierce steel prospecting auger is a tool which can also be used without a derrick to bore test holes from 10 to 50 feet into loose soils or clay. Holes from 2½ to 6 inches in diameter can be drilled and samples obtained. The auger can be turned either by hand-wrenches or by horse-power.

Where the borings are to be of an extensive character a well-drilling machine can be utilized, such as shown in Fig. 314, and which can be run onto an ordinary flatboat and towed to place.

The tools for drilling are a temper screw for regulating the height of the drill, a sinker bar to give the weight, steel jars, and drilling-bits. A sand-pump is used to clean the hole and obtain samples; rope-spears, rope-knives, and fishing-tools to remove lost rope, tools, and pebbles or other obstructions. The drill holes, unless

through rock, are cased with iron pipe which can be withdrawn when the hole is completed.

The borings made by the Mississippi River Commission were

FIG. 314.—STEAM-POWER WELL-DRILLER.

very extensive, and a special tripod apparatus (Fig. 315) was devised with a view to easy transportation and easy repair in the field. The tripod was 30 feet in height, with a strong head or cap, surmounted

by a galvanized-iron guide-pipe 20 feet in height, in two sections, and held in place with guy-ropes. The men operating the tools stood upon the triangular platforms which were attached to the

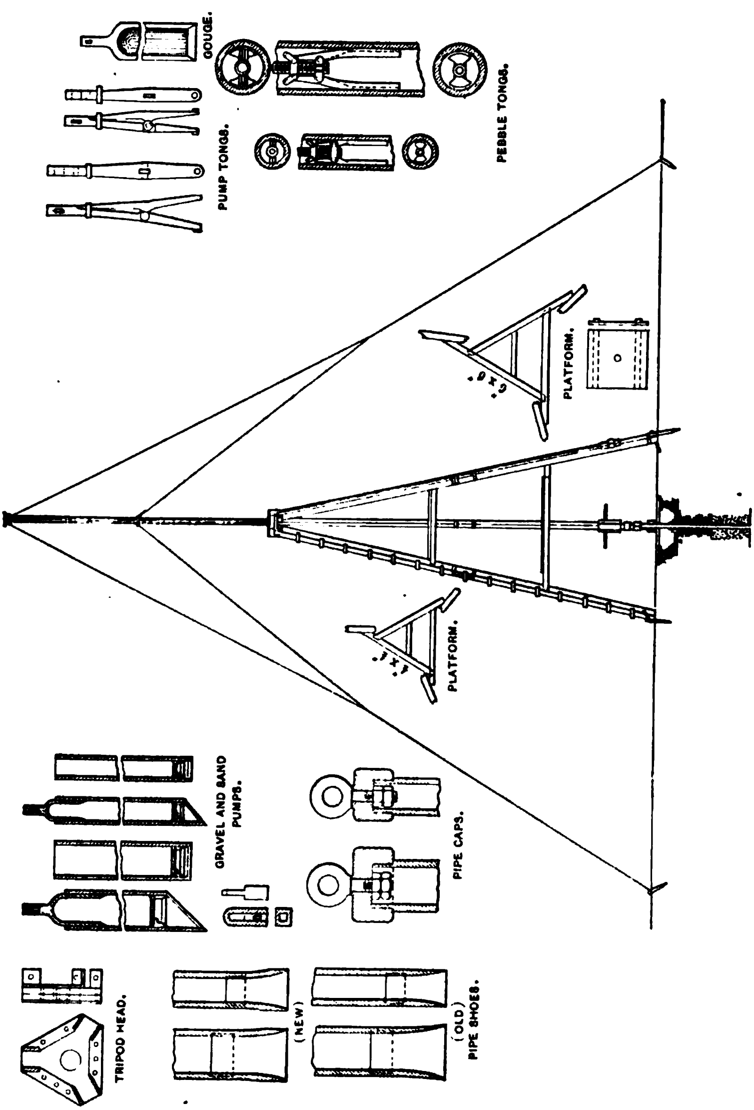


Fig. 315.—Test-boring Apparatus, Mississippi River Commission.

legs. The casing was iron pipe in 10-foot lengths and screwed together so as to present a smooth surface, while the bottom was provided with a steel cutting-shoe, having a mouth slightly larger than the pipe.

The sinking is accomplished by driving and twisting, the driving being done by means of the clamp on the pipe and the maul sliding on the pipe. (Fig. 316). The weight of the maul is from 80 to 100 pounds and is worked by three men giving it a lift of about 2 feet, the best results being obtained when the men act in concert and give rapid blows. The removal of the core and samples is accomplished by means of the various tools shown in Fig. 315, and requires great care and considerable experience. The pump was raised and lowered by means of the reel attached to one leg of the tripod, and its distance from the surface noted from graduations on the pump-rod. When the boring is completed the tube is withdrawn by a system of compound levers, assisted by a set of differential blocks when necessary, as the force exerted was often as much as the strength of the pipe at the joints. The pebble-tongs were for use in removing large pebbles which would not enter the pumps, and for recovering lost tools or the pump itself in case of becoming detached.

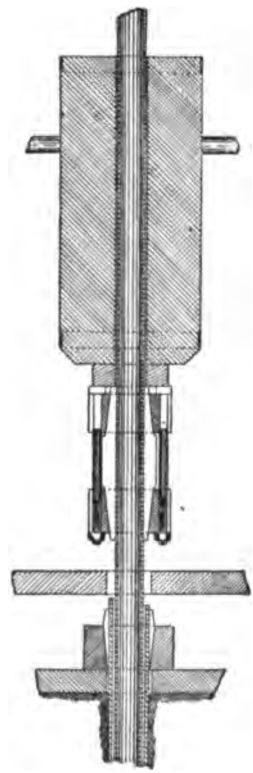


FIG. 316.—CLAMP AND MAUL.

The above account is taken from the report of J. W. Nier, assistant engineer, to which reference must be made for other details.

The poor results obtained in examining the bottom by means of any of the preceding methods of making borings has been mentioned in a number of places in the preceding chapters, and practically the only type of borings that can be relied upon are core borings, made with a diamond drill, or some modification of it.

The borings for the Chicago & Northwestern Railroad Company bridge over the Mississippi River at Clinton, Iowa, were first made by a churn drill, as shown in Fig. 317, where the dotted line was the supposed bed-rock. By the use of a Sullivan diamond drill the bed-rock as shown by the solid line was actually found in some places to be over 30 feet below the original rock profile.

Core borings can be made with a diamond drill, operated by gasoline or steam, Fig. 318, or by hand, Fig. 319. The diamond drill consists of a hollow bit in which are set " black diamonds " or carbon. This

bit is attached to a hollow rod made up in 5- or 10-foot sections, screwed together so that the tool can be lengthened as the depth of the hole increases. About every 8 or 10 feet the drill rod is hoisted out and the core removed, it being caught or held by a core-lifter. This apparatus will operate just as well through the water or soft material as it would to start in on the dry ground surface, so that for testing the bed of a river or the bottom under any body of water, it can be readily used, and will give the exact truth. It is necessary that great care should be exercised in selecting the carbon best suited for this work, and this should be done by the manufacturer or someone having considerable experience. Where the work is not

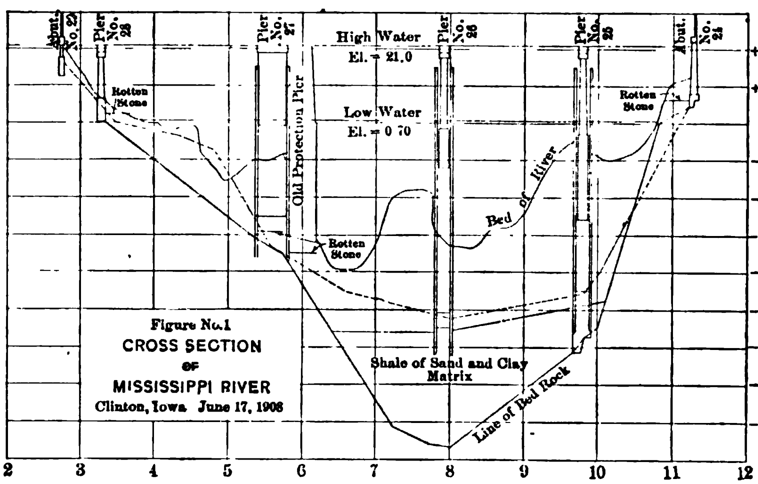


FIG. 317.—TEST-BORING BY DIAMOND DRILLS. C. & N. W. RY.

extensive enough to break in a crew of men, they should be obtained through someone that can furnish at least an experienced operator, although where the work will last some time, a first-class stationary engineer can soon learn to detect when the drill is getting into loose material that will cave, and so avoid trouble. An unskillful operator will often lose the hole or cause a great amount of wear on the tools and break up a lot of carbon. The diamonds, or carbon, must be set in the bit by someone having experience in fitting them up. Work of this character will be undertaken on contract by the manufacturers at varying prices, according to the amount and character of the work.

The cost of the drilling for the Chicago & Northwestern R. R. work already referred to averaged $1.83 per vertical foot. These

cores were 2 inches in diameter, although smaller cores can be taken out if the work is done carefully. Where there is a very extended series of borings to be made, the cost might be reduced to as low as $1.50 per foot, but will range from this to $2.50 to $3 per foot

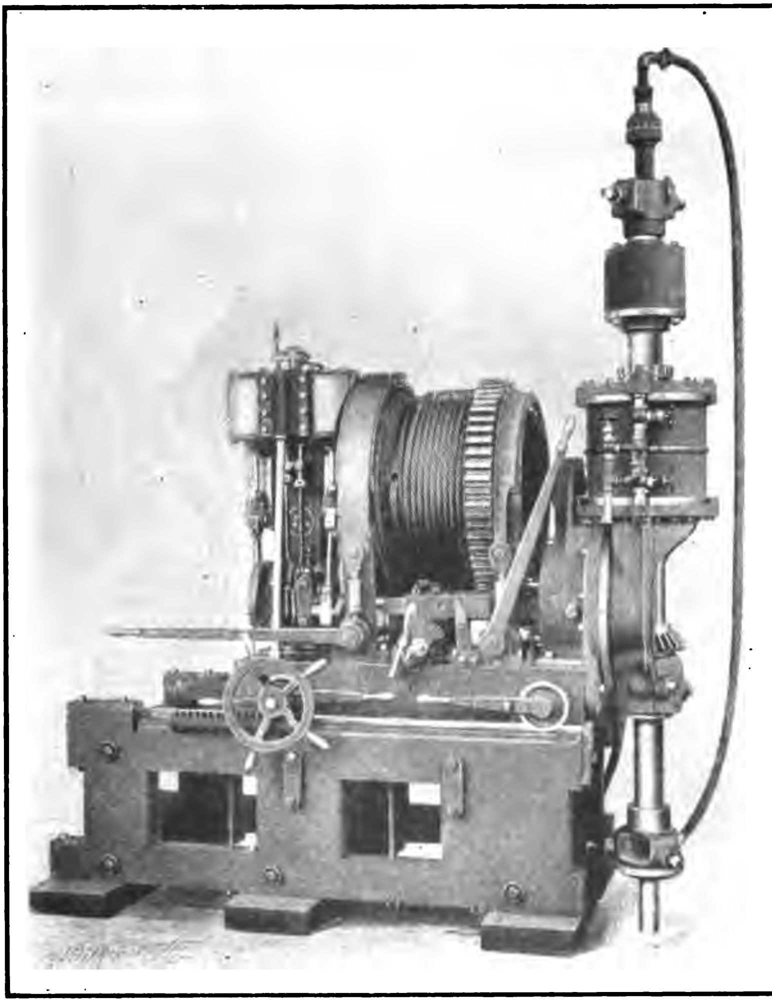


FIG. 318.—SULLIVAN POWER DIAMOND DRILL.

for difficult work; but this is low-priced insurance for knowing exactly what class of material is being dealt with.

The conclusions arrived at by F. H. Bainbridge, Engineer on the Chicago & Northwestern R. R., are as follows:

"The final location of the caisson can be accurately determined by core borings, and cut stone and timber ordered without any waste or delay waiting for material for which no provision had been made.

"The contractor in bidding on the work knows exactly what material is to be encountered, and will make a lower bid when there is no uncertainty. The difference in cost between handling in a caisson, material which can be taken out through the blowpipe, and material which must be locked out in buckets is very great.

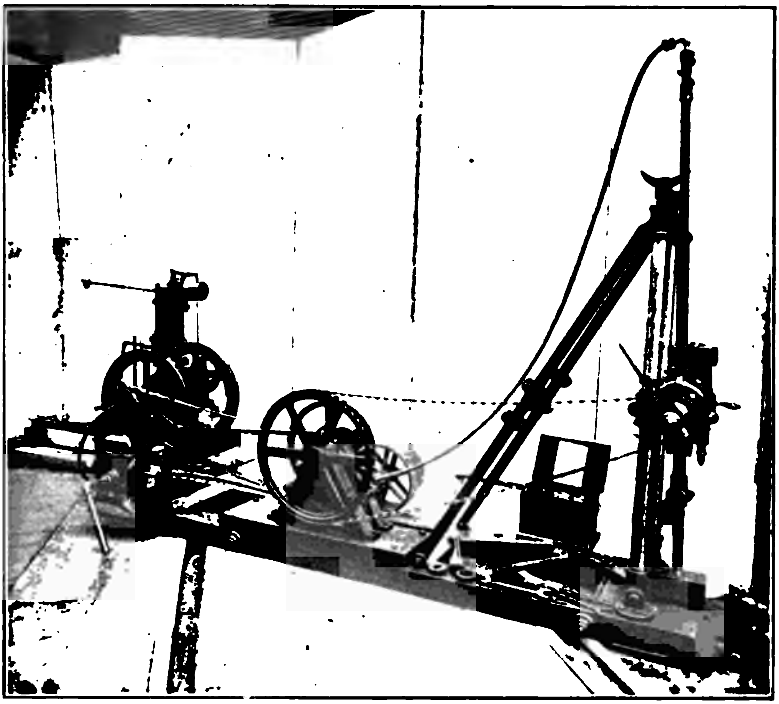


FIG. 319.—SULLIVAN HAND AND POWER DIAMOND DRILL.

"The piers can be located in the most economical position. Often a change of a few feet in locating a pier may make a difference in cost of tens of thousands of dollars.

"Much can be learned as to the character of the foundation that cannot be learned from the interior of the caisson. In limestone formations subterranean caverns are common, and in both lime and sandstone formations overhanging subterranean cliffs are found. The existence of these can be determined with the drill, but cannot be learned from the interior of the caisson."

The McKiernan-Terry core drill is operated with a shot bit instead of a bit set with diamonds, and many records show it to be less expensive for making borings than the diamond drill, cores hav-

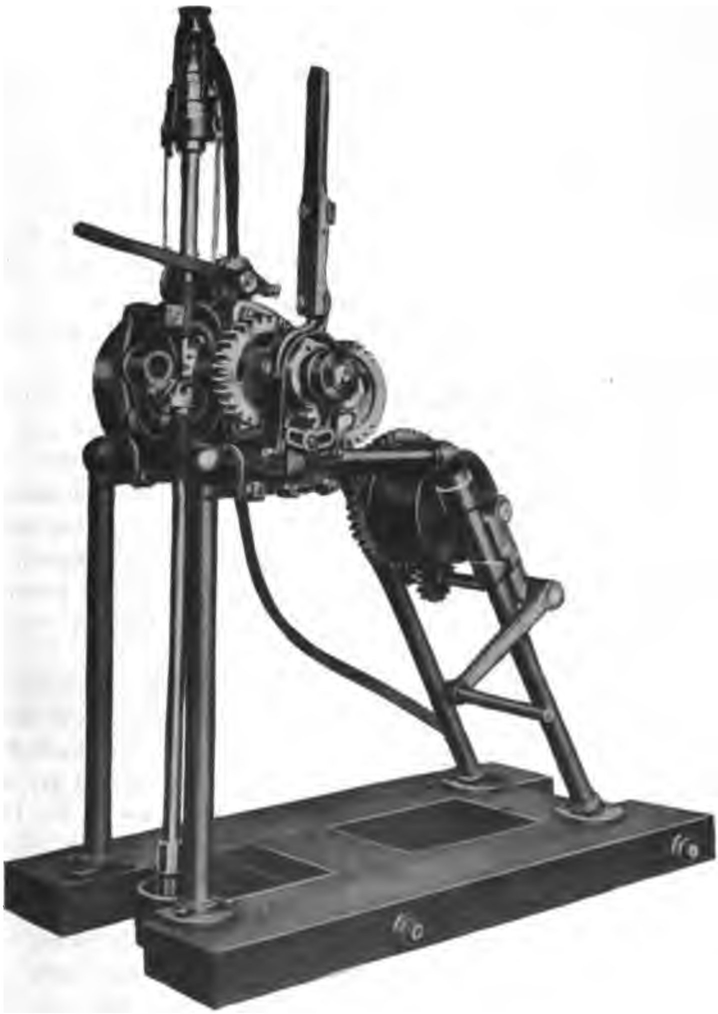


FIG. 320.—MCKIERNAN-TERRY CORE DRILL.

ing been taken out up to 4 inches in diameter at a total cost of less than $1.50 per foot. It is, of course, possible to get better cores from friable material with a large core than with a small one, and perfect cores up to 16¾ inches are taken out with the standard machines,

but they may be fitted with tools up to 60 inches in diameter where it is only necessary to go but a short distance. The class "Z-1" drill is shown in Fig. 320, and the shot bit is shown in Fig. 321. The drilling, of course, is not done with perfect shot, but by the broken pieces which are angular in shape.

FIG. 321.—SHOT BIT FOR CORE DRILL.

The class "Z-1" drill is employed for prospecting and testing work. Its light weight commends its use for such purposes, particularly in places inaccessible to a larger and heavier apparatus. The net weight of the drill shown in the cut on the preceding page is 437 pounds. It can be knocked down and boxed in packages of convenient size for mule-back transportation.

The capacity of the "Z-1" drill is 400 feet, to which depth it will bore a $2\frac{1}{4}$-inch hole and cut a $1\frac{1}{4}$-inch core. Larger tools may be used, which will bore a 3-inch hole to a depth of 250 feet and cut a $1\frac{3}{4}$-inch core. The power to operate this drill is generally supplied by a gasoline motor, to which it is belt-connected, although a steam engine, electric motor, or horse-power may be used if preferred. The drill is fitted with a swivel head, which permits of drilling up to an angle of 45 degrees with the vertical.

On the rear of the frame is mounted a hand hoist, used for raising or lowering the drill rods in the bore hole and for driving casing through soil to bed-rock. Pressure on the drilling tools is applied by means of the sensitive feeding device shown in the cut.

A tripod derrick is generally used with this drill, and, as a rule, it is built from timber cut on the ground. It should be of sufficient height to admit of two lengths, or 20 feet, of drill rods, being raised at a pull. For shallow holes, where but little power is required to rotate the rods, the drill can be arranged for operation by hand. Particulars regarding such equipment will be furnished on application.

When the examination of the site has been completed and the borings finished, the form of foundations may be decided upon, due weight being given to good foundations and to the allowable expenditure. Should the obtaining of good foundations be seen to

be very expensive, long spans must be adopted to require few piers in the river; but if inexpensive, much shorter spans, with more piers, may be used.

The length of spans for a least cost of structure was formerly assumed to be decided when the cost of one span was made equal to the cost of one pier, and for spans of certain capacity this might be approximately true, but a very neat mathematical solution of this problem by Alfred D. Ottewell, consulting engineer, was published in the *Engineering News* of Dec. 14, 1889. The total length of the structure in feet was represented by l, the number of spans by n, the length of one span in feet $l \div n$ by s, the cost of one span in dollars by c, the cost of one pier in dollars by p, the total cost of the structure in dollars by y, while a and b are constants.

From the estimated cost of a large number of spans, a curve of costs was plotted and the following equation of a parabola deduced:

$$c = a + \frac{(s-20)^2}{b}. \qquad (1)$$

Since there are n spans and $n+1$ piers, the total cost of the structure would be

$$y = nc + (n+1)p. \qquad (2)$$

Then by substituting the value of c from (1), reducing and making the first differential coefficient equal to zero, the cost of one pier is obtained, which will make the total cost of the structure a minimum, or

$$p = \frac{s^2 - (ab + 400)}{b}. \qquad (3)$$

Or when the cost of a pier has been estimated, the economical length of span may be found by a transposition of the above formula:

$$s = \sqrt{ab + 400 + pb} \qquad (4)$$

The values of a and b may be found by substituting in equation (1) computed values of the cost of a number of spans for an actual loading. Values of s, p, and c may then be computed and tabulated for spans from 100 feet upwards, as formula (1) is not true for shorter lengths.

In an actual calculation for B. & O. R.R. loading, which consists of two 125-ton engines followed by a 4000-pound per lineal foot train-load, a was found equal to 1950 and b to 3.05. Assuming a case where the length of the bridge is 700 feet, where the height of the piers will average 25 feet, and the average cost of piers and abutments be $4310, then from formula (4) the economical span will be found equal to 160 feet. The total cost of the structure will be found, by using formula (1), and the cost of piers as above, to be $59,700; while with only four spans of 175 feet the total cost would exceed $60,800, and with six spans of 117 feet would be about $61,400.

Should there be any doubt as to the ease of obtaining foundations, the prudent engineer might deem it wise, however, to build the four-span structure and avoid the risk and delay which would be caused by another foundation in the river.

After deciding upon the number and location of the piers, they must be designed with reference both to their being as slight obstructions to the water as possible and to their architectural appearance.

Particular attention was given to the design of piers by the late Geo. S. Morison, consulting engineer, whose work on the bridges across our great rivers is notable for its strength, simplicity, and finished appearance. In a lecture he described the process of the design of some large piers: "Fourteen years ago I had occasion to design a bridge pier for a bridge across one of our Western rivers, and I tried to make an ornamental pier. When the plans were completed I did not like them. One change after another was made, all tending to simplicity. Finally the plans were done. From high water down, the pier was adapted to pass the water with the least disturbance; it had parallel sides and the ends were formed of two circular arcs meeting. Above high water the ends were made semicircular instead of being pointed. The pier was built throughout with a batter of one in twenty-four. A coping 2 feet wider than the body of the pier projected far enough to shed water, and the projection was divided between the coping and the course below. Another coping with a less projection surmounted the pointed ends where the shape was changed. It was as simple a pier as could be built, and in every way fitted to do its duty. I had started to make a handsome pier. The pier that was exactly what was wanted for the work was the only one that satisfied the demands of beauty. Forty-three piers of precisely this design, no change having been made except in the varying dimensions required for different structures, besides eight others in which only the lower parts are modified, are now standing in eleven different bridges across three great Western

rivers. In designing a pier it must be remembered that the portion of the pier below the water has more to do with the free passage of the water than that above water. In a deep river the model form of the pier should begin near the bottom of the river and not at low water. Many rivers in flood time carry a great amount of drift. A pier like that which I have described catches but little of this drift. If, however, a rectangular foundation terminates but little below water, that foundation may both disturb the current and catch the drift."

The piers of the Omaha bridge which carries the Union Pacific across the Missouri River are illustrated in Fig. 322, and were constructed as described and are among the most beautiful piers in this country.

In Europe, where money is more lavishly expended on great works of engineering, piers of great architectural beauty are much more frequent. The Russian Government railways, which have seemingly been constructed without regard to expense, have many beautiful examples of bridge masonry and piers; the view of one of them (Fig. 323), with curved ends, shows the elegant and massive character of the masonry. While extremely simple in design, the cut-stone coping and the molded corbel course below give it a finish which cannot be surpassed.

The design of piers for strength and stability is fully treated in Baker's "Masonry Construction," but some experiments, which were made with reference to the proper form to occasion the least resistance, will be quoted at length from Cresy.

The introduction of piers into a channel gives rise to a great disturbance in the velocity and flow of the water. Rapid currents are formed which cause the bed of the stream to become washed and the foundations to be endangered; eddies are created which are likewise undesirable, and it becomes necessary to adopt such a form for the ends of the piers that the disturbance to the flow shall be small.

M. Bossut, in a French work on jetties, thought to have solved this problem by mathematics, his conclusion being that the starling should be triangular, the nose being a right angle.

M. Dubaut, in his "Principles of Hydraulics," gave another solution which was more nearly the truth, in that he arrived at the conclusion that the faces of the starling should be convex curves. The true form is most nearly reached when these curves are tangent to the sides of the pier, and, further than this, regard must be paid to giving enough solidity to the starlings to protect them from ice

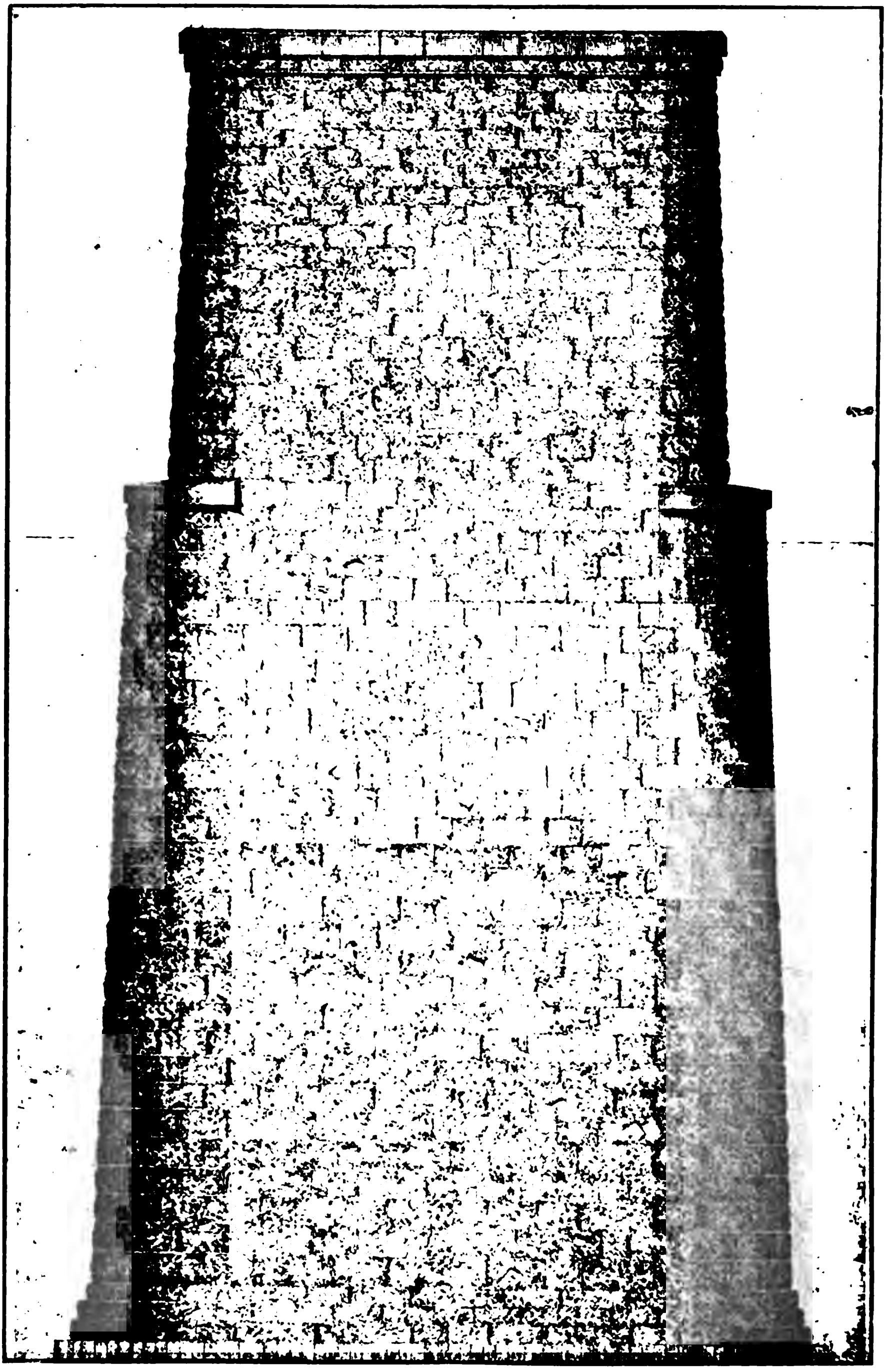


FIG. 322.—PIER OF OMAHA BRIDGE, UNION PACIFIC SYSTEM.

and drift. A happy medium would seem to be reached by making the curves with a radius equal to one-sixth of the circumference, described on the sides of an equilateral triangle.

Experiments were made with models of different forms, which were placed in a rectangular canal between boards of 50 centimeters in length, in which the water flowed about 40 millimeters in height, the models being 15 centimeters in thickness. By means of a fall,

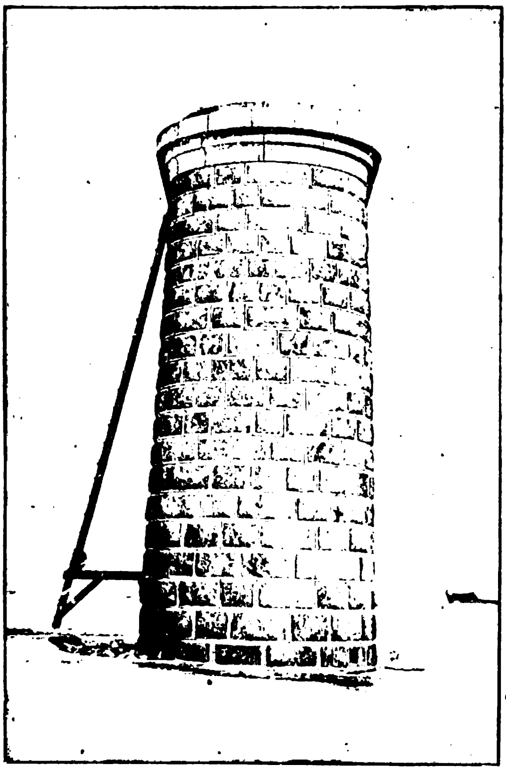


FIG. 323.—RUSSIAN PIER, RUSSIAN STATE RAILWAYS.

the water was given a velocity of 3 meters 9 centimeters per second, the contraction, eddies, and currents being carefully measured. The first experiment was made on a pier (Fig. 324 *a*) with rectangular starling. An eddy was formed before the pier 34 millimeters high, in a nearly circular band A, falling nearly vertical at the corner. There were two other currents along the faces of the pier, the height of which can be seen in the cross-sections.

The second experiment (Fig. 324 *b*) was with a triangular star-

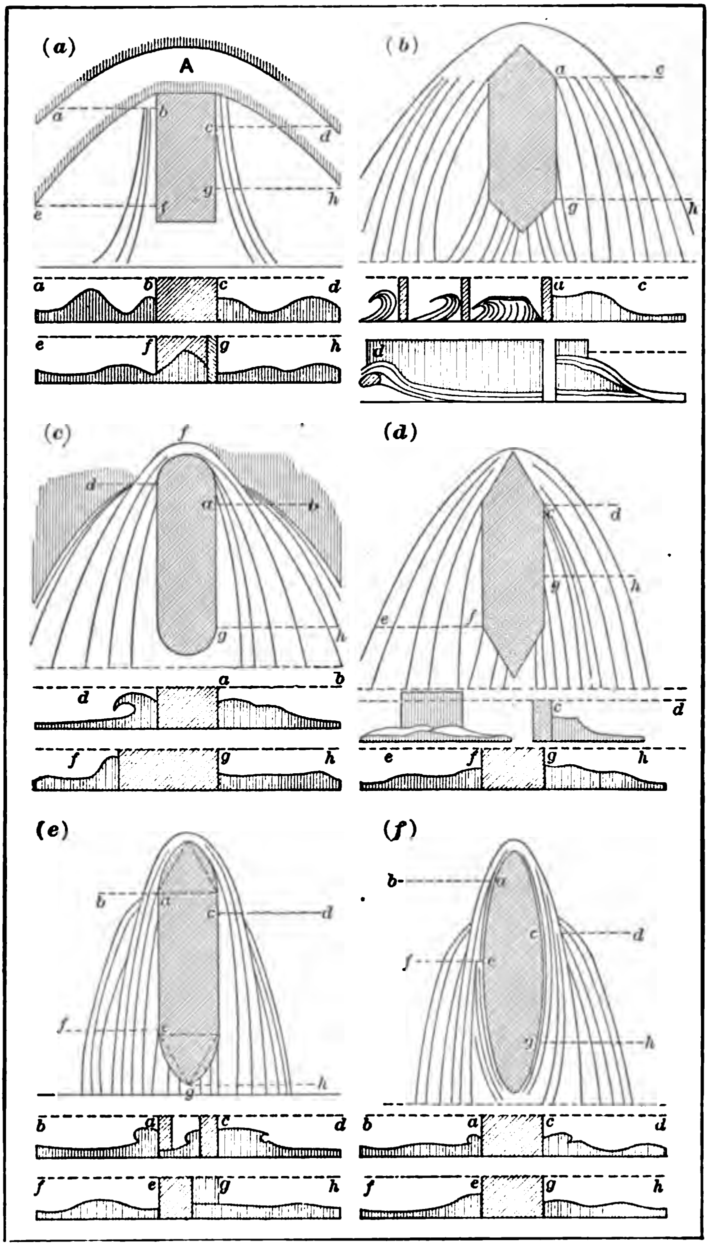


FIG. 324.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

ling, the nose being a right angle. It formed a less obstruction than the square end, but the fall at the shoulder was as deep and more dangerous, while eddies were formed as seen in the sections.

The third one (Fig. 324, *c*) had a semicircular starling. The eddy was not so wide, but nearly as high.

The fourth model had a triangular starling, with an angle of 60° at the nose (Fig. 324, *d*). The eddy was less, as was also the fall at the shoulder.

The starling in the fifth was formed by two circular arcs, tangent to the sides and described on the sides of an equilateral triangle (Fig. 324, *e*). The eddy was small and there was no fall at the shoulder.

The sixth (Fig. 324, *f*) was a model the plan of which was an ellipse, of which the small diameter was one-fourth the length, and the eddy was less than any of the others.

The seventh model (Fig. 325 *a*) had a starling with concave faces, such as is sometimes used where the wing-wall meets an abutment. It produced the most dangerous currents of all.

The eighth (Fig. 325 *b*) was of the same form as Fig. 324 *e*, but the water was supposed to mount the springing of the arch.

The ninth and tenth experiments (Figs. 325 *c* and 325 *d*) were on the same forms as Figs. 324 *e* and 324 *f*, but the current had a velocity of 4 meters 87 centimeters per second, such as a river would have in its overflow. The eddy (Fig. 325 *c*) rose to nearly twice the height, as was the case with the lesser velocity, and, while there was no fall, the inclination formed along the faces was more rapid.

The effect with this velocity on the elliptical pier (Fig. 325 *d*) was similar to the lesser velocity, but more marked. It may thus be concluded that the elliptical section offers the least resistance to the current and occasions the least contraction, while the form with convex starling comes next, and of piers with triangular starlings the one with the 60° nose is the best.

Where ice is to be provided for, the nose is often inclined to allow large cakes to mount it and break in two, without doing further damage. For any large or important structure, the design of the piers should receive a great deal of study, and be designed not only with reference to their theoretical form, but with reference to the form of pier which has shown the best results practically and has been found to be best suited to the velocity of the stream in which they are to be built, and to best withstand the drift and ice that may be met, with giving at the same time all the consideration

possible to the architectural effect and to the harmony with the entire structure.

The piers of the Knoxville steel arched cantilever, Figs. 326–7, were equally spaced by the author in designing the structure, in order to make the bridge symmetrical and a good piece of architecture.

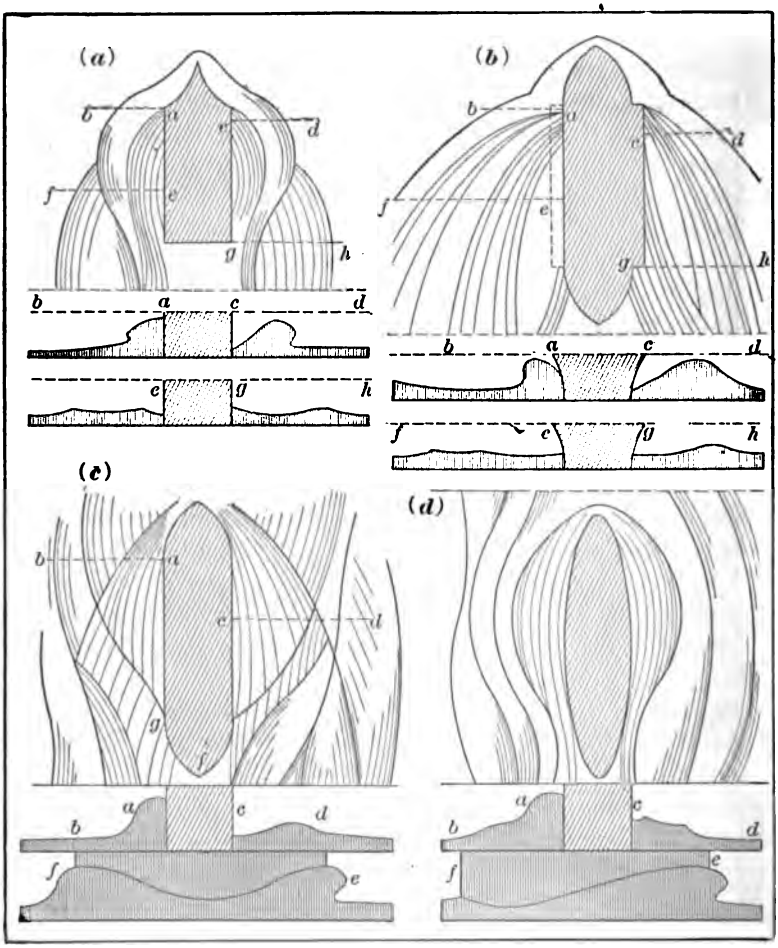


FIG. 325.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

The piers had a cross-section as in Fig. 325 *c*, as the current in the river was very swift during high water. Had more money been available to add ornamental corbels and copings and proper capping around the steel shoes and bolsters, the appearance would have been much improved.

Fig. 326.—Knoxville Steel Arched Cantilever.

FIG. 327.—KNOXVILLE STEEL ARCHED CANTILEVER DURING ERECTION.

CHAPTER XXIII

LOCATION AND DESIGN OF PIERS (CONTINUED)

THE stones most used in the building of piers are granite, sandstone, marble, and limestone, the amount of granite used for this purpose being about the same or possibly a little less than sandstone. Next to these, limestone is used most largely and marble the least of the four, owing to the fact that most marble is suitable for dressed stonework for buildings and too expensive for use in piers.

Granite is most largely produced in the New England States, and notably in Maine, Massachusttes, and Vermont. Next to these States comes California, with about as much of an output as Vermont. Only fifteen of the States in the Union do not have an output of granite, so that it may be said to be available at greater or less cost in any part of the United States.

Sandstone is most largely produced in Pennsylvania, Ohio, and New York, although but thirteen of the States are non-producers of this most commonly used building material.

Marble is most largely produced in Vermont, although New York, Tennessee, and Georgia have extensive quarries and are large producers.

Limestone is extensively quarried in Pennsylvania, Ohio, New York, Missouri, Wisconsin, Illinois and Indiana, only six States being without output of this stone.

Granite is the best building stone for use in constructing piers, and under this head are included the true granites, gneiss, mica-schist, andesite, syenite, and quartz-porphyry.

Sandstone covers all the consolidated sands, the strength of sandstone depending entirely upon the cementing material, as it is simply quartz grains cemented together, and when silica is the cementing material it is the very best. Quarrymen term sandstone according to its quality as bluestone, freestone, and conglomerates.

Limestone is usually the least valuable of stone that is used in building piers, owing to the fact that it is too soft and stands the weather poorly. It consists practically of amorphous calcium car-

bonate, sometimes cemented together by crystalline substances with many impurities. Under the head of limestone is also usually included magnesium carbonate, called magnesium limestone, and when the stone is about equally made up of carbonate of lime and magnesium it is termed dolomite. Dolomite is much harder than ordinary limestone, and consequently forms a much better building material. Chemically, there is little difference between limestone and marble, except that marble has been crystallized by the action of heat.

Of the quality of the granite to be obtained in any part of the United States very little need be said, as it is always a desirable building material, and only the question of cost comes up for consideration as to whether it can be used or not. In the New England States, Georgia, California, and Washington it is cheap enough to make it possible to use it wherever a reasonable amount of money is available for the construction of bridge piers or foundations of any sort. The softer sandstones obtainable throughout the country and which have a compressive strength of about 5000 pounds per square inch are suitable for building bridge piers where not too much ice or abrasive action of any character is to be contended with.

The freestones of northeastern Ohio, and similar stone wherever found throughout the United States and similar to the Chuckanut stone of Puget Sound, are almost as desirable as granite for the construction of piers.

Limestone does not very often occur of such quality as to warrant its use in masonry construction, and while it may seem to be of sufficient hardness to warrant consideration, attention need only be called to its use in the State Capitol building of Ohio, where it has weathered badly, to make it seem advisable to find some other material if possible.

Where marble is plenty, it is of course of sufficient strength and hardness to make it desirable for use in foundation work and for piers, provided the cost is not excessive, and a poorer quality of marble found in Tennessee and Georgia, and known as iron limestone, is certainly one of the best building stones to be found anywhere. The most famous limestone to be found in the United States is the oolitic limestone of Indiana. It is very easily quarried and hardens on exposure to the air, so that it is quite durable; and, on account of the large size of the blocks in which it can be gotten out, is very much used for massive work.

One of the things least often considered in the selection of stone for ordinary piers is the color; although, in the case of piers for

city bridges, towers for suspension bridges, or work of this character it is very desirable to have the material of pleasing appearance, and this is most readily found in the granites and marbles. The ordinary gray or bluish gray of many of the sandstones is also very pleasing, and in many cases other colors can be found for belt courses, copings, and trimmings of various kinds.

While the color of stone is apt to change considerably after it is quarried, it is usually possible to know what the change will amount to, by seeing stone of the same kind that has been in use. The gray color of many of the granites is due to a mixture of light feldspar with a dark-colored mica and very fine hornblende. The sedimentary rocks are colored with iron and various other minerals, and it is always necessary when iron is the coloring-matter to make sure that in weathering the stone will not turn rusty.

One of the most important qualities to be taken into account in selecting building stone is its durability under changes of temperature and abrasion, although the stone may prove valueless owing to chemical changes due either to its going to pieces by the action of the water, carbon dioxide, or some of the organic acids. The change of temperature affects rock by the unequal expansion and contraction of the various minerals composing it, or the rock may be so porous as to become filled with water, and when this freezes the expansion of the ice will cause it to crack or break. The report on the building stones of Wisconsin states that " the expansive force of heat is well shown in many of the limestone quarries of Wisconsin, where beds from 5 to 6 inches in thickness are for the first time exposed to the heat of the summer sun. These thin beds become heated throughout their entire thickness, and arch up on the floor of the quarry, generally breaking and completely destroying the stone." The effect of freezing and thawing is well stated in Vol. II of the Washington Geological Survey as follows: " All rocks are more or less porous; and these pores, before the rock is quarried, always contain more or less water, and after being quarried for some time and exposed to the atmosphere they lose this water. However, when rain-storms occur they are apt to absorb more water, and if the temperature falls below the freezing-point when the stone is in this condition the water will be frozen. As is well known, water on freezing expands and in expanding exerts a pressure or expansive force equal to about 150 tons to the square foot. It is plain to see that if the rock contains any very large amount of water and freezes, the result will be the spreading apart or separating somewhat of the particles composing the stone. Then the water thaws and freezes

again, and so on indefinitely, and the result is that particles are finally completely loosened and fall out. This effect is principally on the surface of the rock and is the cause of the scaling frequently seen in buildings that are built of certain kinds of stone. In addition to the pores which occur in the rocks, there are the openings which occur along the joint, bedding and foliation planes. Water falling on the surface of the ground and more or less of it sinking into it enters these cracks or crevices; and while it will flow more readily along these than it does through the pores of the rock, still many times these will be filled and freeze while in that condition. As has already been stated, water when freezing exerts a very great expansive force and will tend to separate the rocks along these planes, and when the ice melts the rocks do not come back to their original position, but retain the position they had when the water was frozen in them. Then these cracks are filled again and refrozen, and the seams opened a little farther, and the same process repeated time after time finally produces a perceptible effect and tends to weaken the stone. This is especially true of sedimentary deposits such as sandstones, particularly where they have marked bedding planes."

The effect of ice, driftwood, and the like is to abrade the surfaces of piers so that with stone that is at all soft it becomes a very serious matter. The action of sand carried by wind-storms, while very destructive, can hardly be considered in the design of piers, as it is seldom that they are so situated as to be affected in this way.

The mineralogical composition of stones has a very important bearing upon their durability, but, as this is fully treated in works on mineralogy, it will not be gone into here.

Chemical and microscopic examinations are often of value, and for any large piece of work, or where the stone as proposed for use has not been used for any great length of time, these tests should be made; but, as a general rule, the physical tests are of very much greater value. Nearly all of our universities and many of the larger engineering offices now have testing-machines (Fig. 123), so that tests can easily be made. They should be carried out by standard methods, so that they will be of value for tabulation with the results of other investigators, and of use to future consumers of the stone.

It was formerly the custom to make tests on 1-inch cubes, but wherever the testing-machine is large enough they should be not less than 2-inch cubes, having 4 square inches of area, or larger if possible, as stone in large pieces has greater resistance per square inch, and thus the actual strength of the stone will be more nearly determined.

The Wisconsin report divides the physical tests into two divisions: First, strength tests, comprising crushing strength, transverse strength—giving the modulus of rupture and the coefficient of elasticity; second, durability tests, covering specific gravity, porosity, weight of the stone per cubic foot, effect of extreme heat, effect of alternate freezing and thawing, action of carbonic-acid gas, and the action of sulphurous-acid fumes.

The crushing strength has usually been considered all that is necessary, but the above report speaks of this test as follows: "It has been computed that the stone at the base of the Washington Monument, the highest structure in the world, sustains a maximum pressure of 22.658 tons per square foot, or 314.6 pounds per square inch. Certain contractors require a stone to withstand twenty times the pressure to which it will be subjected in the wall, while others only require ten times that pressure. Even if requiring a factor of safety of twenty, the strength required for a stone at the base of this monument would be only 6292 pounds per square inch. The pressure at the base of our tallest building can scarcely exceed one-half that at the base of the monument, or 157.3 pounds per square inch. According to the above estimate, stone used in the tallest buildings does not require a compressive strength above 3146 pounds per square inch. There is scarcely a building stone of importance in the country that does not give a higher test than this. Ordinary building stone has from two to ten times the maximum required crushing strength. A stone having a crushing strength of 5000 pounds per square inch is sufficiently strong for any ordinary building." So that it will be seen that very few stones will not be strong enough in this regard.

The following tables, which are taken from the Washington Geological Survey, give a large number of tests which have been made on building stone in various parts of the country:

TABLE XLIII.—CRUSHING STRENGTH IN POUNDS PER SQUARE INCH, SPECIFIC GRAVITY, AND RATIO OF ABSORPTION OF BUILDING STONE

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
GRANITE.			
(1)*Montello, Wisconsin	43,973	2.639	.079
(1) Granite City, Wisconsin	25,000	2.675	.133
(1) Berlin, Wisconsin	32,747	2.643	.143
(1) Granite Heights, Wisconsin	16,723	2.631	.180

* See p. 452 for references.

TABLE XLIII.—*Continued.*

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
GRANITE—*Continued.*			
(2) East St. Cloud, Minnesota	28,000	2.692	2.59
(2) Sauk Rapids	21,500	2.710	.190
(2) Beaver Bay, Minnesota	20,750	2.69	.140
(3) Fourche Mountain, Arkansas	29,000	2.642	1–1673
(3) Fourche Mountain, Arkansas	28,700	2.635	
(3) Fourche Mountain, Arkansas	21,500		
(4) Little Rock, Arkansas	22,388		
(4) Little Rock, Arkansas	17,407		
(4) Millbridge, Maine	19,917		
SANDSTONE.			
(2) *Hinckley, Minnesota	19,000	2.470	4.88
(2) Dresbach, Minnesota	6,500	2.380	11.48
(2) Jordan, Minnesota	4,750	2.340	12.69
(1) Ablemans, Wisconsin	13,669		
(1) Ablemans, Wisconsin	11,030		
(1) Ablemans, Wisconsin	8,602		
(1) Ablemans, Wisconsin	10,056		
(1) Dunnville, Wisconsin	2,502	2.601	15.130
(1) Port Wing, Wisconsin	5,498	2.638	10.330
(1) Houghton, Wisconsin	4,549		
(1) La Valle, Wisconsin	13,350		
(1) Bayfield, Wisconsin	4,588	2.639	4.760
(5) Birdsboro, Pennsylvania	11,448		
(5) Waltonville, Pennsylvania	14,000	2.350	1–27
(5) Waltonville, Pennsylvania	12,730		
(5) Lumberville, Pennsylvania	22,250	2.660	
(5) Laurel Run, Pennsylvania	17,600	2.660	1–900
(5) White Haven, Pennsylvania	29,252		
(5) Portland, Connecticut	12,580	2.350	1–40
(5) Middletown, Connecticut	6,250	2.360	1–40
(5) E. Longmeadow, Massachusetts	12,330	2.480	
(5) Medina, New York	16,031	2.400	1–53
(5) Marquette, Michigan	6,150		
(6) St. Anthony, Indiana	3,000		3/40
(6) Riverside, Indiana	6,100		1/25
(6) Riverside, Indiana	6,800		1/50
(6) Worthy, Indiana	6,825		
(6) Berea, Ohio	11,213	2.110	1/20
(6) Hummelstown, Pennsylvania	12,810		
(6) Gunnison, Colorado	5,250	2.200	.09
(6) Cleveland, Ohio	6,800	2.240	1/37
(6) N. Amherst, Ohio	5,450	2.140	1/99
(6) Angel Island, California	4,574	2.730	2.730
(6) San José, California	2,400	2.640	1/16
(6) Bass Island, Wisconsin	4,850		

* See p. 452 for references.

The engineer should always make careful inquiries as to whether the manner of quarrying stone injures it in any way, as the use of

explosives will very materially shatter many kinds of stone and practically ruin them for heavy construction work.

Most all quarrying (Fig. 328) is now done by the use of drills or channeling-machines, so that damage from explosives is not so frequently found as formerly.

The larger granite quarries use no machinery in quarrying the stone other than rock drills and hoisting-apparatus. Lewis holes are drilled close together in groups of two or three, the partitions broken down, and when a series of these have been drilled around the piece to be blasted out, explosives are used to loosen the rock, and by this method the action is wedge-like, and the rock is not damaged.

TABLE XLIV.

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
LIMESTONE.			
(1)*Knowles, Wisonsin	29,189	2.793	1.76
(1) Bridgeport, Wisconsin	10,112	2.740	5.49
(1) Bridgeport, Wisconsin	6,675	2.740	5.46
(1) Duck Creek, Wisconsin	23,783	2.843	.419
(1) Sturgeon Bay, Wisconsin	31,957	2.841	.19
(1) Sturgeon Bay, Wisconsin	39,983	2.700	.64
(1) Genesee, Wisconsin	36,731	2.833	1.10
(1) Genesee, Wisconsin	29,253	2.829	1.15
(1) Marblehead, Wisconsin	42,787	2.856	.31
(1) Lannon, Wisconsin	31,936	2.814	1.32
(1) Fountain City, Wisconsin	8,830	2.804	4.95
(1) Wauwatosa, Wisconsin	19,111	2.821	2.29
(1) Wauwatosa, Wisconsin	13,406	2.826	2.53
(1) Wauwatosa, Wisconsin	23,744		
(2) Red Wing, Minnesota	23,000	2.750	2.95
(2) Stillwater, Minnesota	10,750	2.590	2.19
(1) Kasota, Minnesota	18,500	2.640	2.51
(2) Mantorville, Minnesota	9,500	2.650	5.37
(2) Minneapolis, Minnesota	21,750	2.770	2.36
(7) Ellettsville, Indiana	6,700		1–31
(7) Ellettsville, Indiana	5,900		1–41
(7) Salem, Indiana	11,700	2.510	1–31
(7) Salem, Indiana	6,900		
(7) Bloomington, Indiana	4,200		1–14
(7) Bloomington, Indiana	5,700	2.460	1–19
(7) Bloomington, Indiana	8,000		1–33
(7) Romana, Indiana	7,000	2.480	1–39
(7) Bedford, Indiana	4,609	2.470	1–23
(7) Bedford, Indiana	14,000		
(7) Bedford, Indiana	6,500		1–24
(7) Salem, Indiana	8,900	2.510	1–31

* See p. 452 for references.

TABLE XLIV.—*Continued.*

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
MARBLE.			
(1) Rutland, Vermont	11,892		
(1) Rutland, Vermont	13,864		
(1) Mountain, Vermont	12,833		
(1) Sutherland Falls, Vermont	16,156		
(1) DeKalb, New York	13,733		
(1) DeKalb, New York	10,478		
(1) DeKalb, New York	12,004		
(1) DeKalb, New York	13,772		
(1) Colton, California	17,783		
(1) Canaan, Connecticut	5,812		
(8) St. Joe, Arkansas	17,835	2.712	0.34
(8) St. Joe, Arkansas	10,447	2.697	0.33
(8) St. Joe, Arkansas	11,265	2.707	0.25
(8) Marble City, Arkansas	8,894	2.691	0.57
(8) Marble City, Arkansas	10,381	2.689	0.49
(8) Rhodes Mill, Arkansas	14,400	2.711	0.29
(8) St. Joe, Arkansas	6,728	2.693	0.37
(8) St. Joe, Arkansas	6,935	2.675	0.56
(8) Montgomery County, Pennsylvania	13,700		
(8) Dorset, Vermont	7,612	2.635	0.58
(8) Cararra, Italy	12,156	2.690	
(9) Georgia	10,000		
(9) Georgia	13,100	2.763	
(9) Georgia	11,400	2.717	
(9) Georgia	12,000	2.707	
(9) Georgia	10,900	2.734	
(9) Georgia	10,800		

(1) Wisconsin Geological and Natural History Survey, Bulletin No. 4, Building and Ornamental Stones, pp. 309–403, by E. R. Buckley.
(2) Geol. and Nat. Hist. Sur. of Minn., final report, Vol. I, pp. 196–200.
(3) Ann. Rep. Ark. Geol. Survey, Vol. II, 1890, pp. 44 to 50, by J. F. Williams.
(4) Tests of Metals, Government Rep., 1905, pp. 319-320.
(5) Appendix Ann. Rep., Pa, State College, 1896, p. 30 (Brownstones),
(6) Twentieth Ann. Rep., on the Geology and Natural Resources of Indiana, p. 323.
(7) Twenty-first Ann. Rep. on the Geology and Natural Resources of Indiana, pp. 313–315.
(8 (Ann. Rep. Ark. Geol. Survey, Vol. IV. 1890, p. 210, by T. C. Hopkins.
(9) Geological Survey of Georgia, Bulletin No. 1, p. 81.

In marble-quarrying channeling-machines are used, which are moved back and forth on narrow tracks and cut vertical channels 5 or 6 feet or over in depth, and a little over an inch in thickness. Some of these machines are so arranged that a channel is cut on each side at the same time. When these channels have been cut, holes are cut horizontally across the bed and the stone split loose by wedges.

Sandstone-quarrying is carried on considerably by channeling-machines, although very many quarries are still operated by blasting out large blocks and cutting them to shape afterwards, although

FIG. 328.—GENERAL VIEW OHIO FREESTONE QUARRY.

some damage may result to the stone from the force of the blasts. The method of working quarries is stated quite fully in the Washington Geological Survey as follows:

"The quarrying of marbles, limestones, and some sandstones at the present time is done quite largely by the use of channeling-

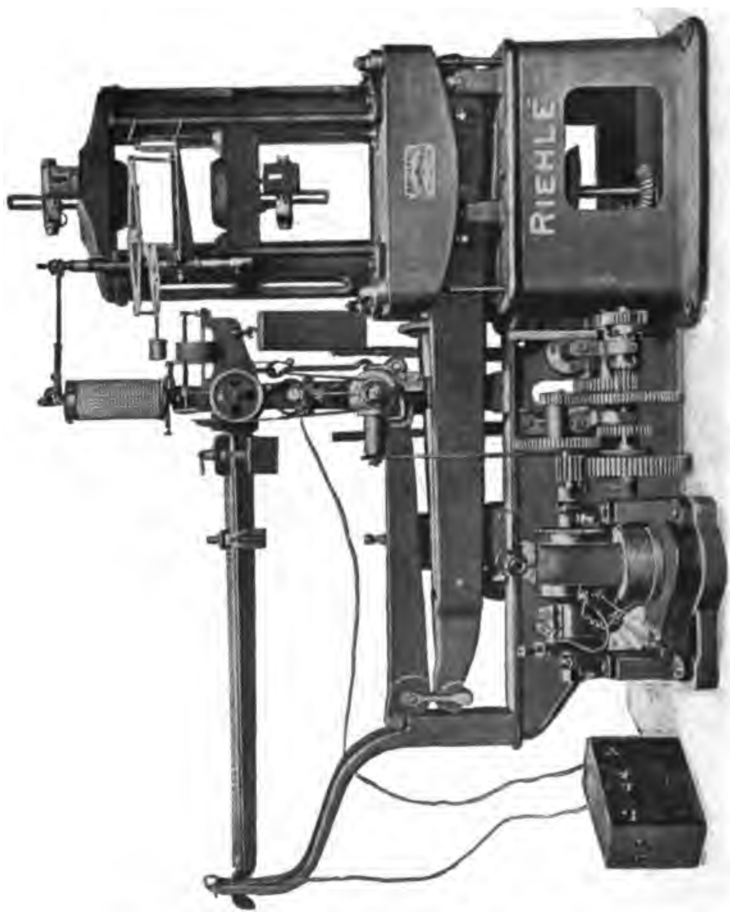


FIG. 329.—TESTING-MACHINE.

machines of some kind (Fig. 330), while, in the harder igneous rocks such as granite, explosives are quite largely used for breaking the rock loose, after which the large masses are split and worked into sizes by hand. In the opening of a quarry in which channeling-machines are to be used, the usual thing to do is to remove the débris

overlying the stone to be quarried and secure a comparatively level floor of the same size that it is desired to make the quarry. When this is done the channeling-machine is put to work and a series of channels the required depth and distance apart are cut, and one of

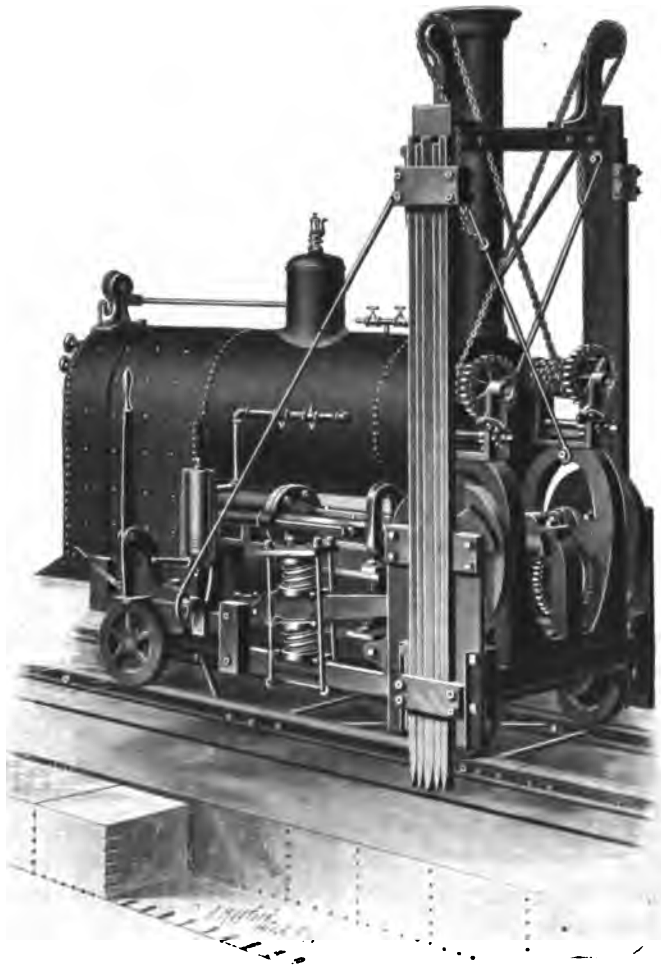


FIG. 330.—WARDWELL CHANNELER.

the blocks loosened on the under side in some manner, usually by wedging, and then lifted out, or it may be removed by blasting. After first block is removed the others may be loosened on the under side either by gadding or by means of wedges. Then another layer is begun and removed in exactly the same manner, and in this way

the quarry floor may be carried down almost any depth provided the stone continues.

"In some quarries what is known as the step or bench system is used and consists in having a ledge of varying width at the back wall each time instead of taking out an entire layer of the quarry floor. This will give to the back part of the quarry the appearance of a set of steps. If the quarry is to be worked after this plan the bar-channeler is probably the best one to purchase, as it is much more easily moved from bench to bench. In the case of quarries worked by hand, either one of the above plans may be followed.

"While many machines have been invented for cutting and dressing stone, still the same slow hand processes that were in use hundreds of years ago are still quite largely used. Large masses of the stone are loosened by means of powder and then these are split into blocks of the required sizes by what is known as the plug-and feather method. This method consists in drilling a series of holes about three-fourths of an inch in diameter and a few inches deep along a line where it is desired to split the stone. Into each one of these holes are placed two pieces of soft half-round iron called 'feathers,' and between these a steel wedge or 'plug' is placed. The quarryman then takes a hammer and moves along this line, striking alternately each one of these wedges until the stone splits and falls apart along this line. There is considerable knack in the splitting of various kinds of stone and it consists simply in being able to take advantage of the rift and grain of a stone, and it is surprising how readily some persons will work a stone into the desired shape, while others can hardly work it into any shape at all.

"In some cases stone is cut to the proper sizes in the quarry by means of channelers, steam-drills, and portable saws, but in most cases marbles, limestones, and sandstones are cut into the desired shapes after leaving the quarry and going to the mill. Usually the stone is taken out of the quarry in large blocks and then taken to the mill, where it is usually cut into the required dimensions by means of saws, and if it is to be carved or polished this is done here, and, in fact, the stone is finished ready for its place in the building.

"Most of the cutting to sizes is done by sawing (Fig. 331). This sawing is done principally by means of gang-saws which consist of a number of toothless blades of soft iron fastened in a frame in a horizontal position and this frame so arranged that it can be moved backward and forward continuously. The stone to be sawed is brought under these saws and the blades set for the required thick-

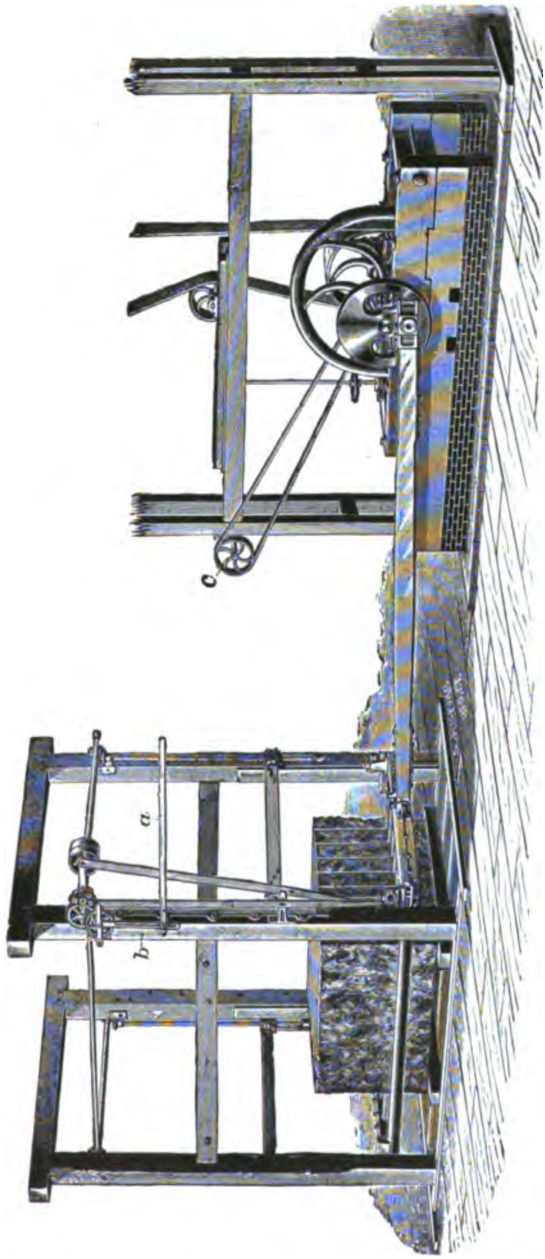


FIG. 331.—STONE SAWING-MACHINE.

ness of the stone and then the machine set in motion. The cutting is done principally by sand or some substitute for it which, along with water, is supplied to the saw-blades. The water softens the stone, and aids in carrying the sand to the saw. The saw may be of almost any length and the frame may contain any number of blades. The blades are usually about $\frac{1}{8}$ inch in thickness and about 4 inches wide. In the latest patterns the frames are lowered automatically as the saws cut into the stone.

"The rate of cutting by these saws varies with the stone, being much faster in some kinds than in others, as, for instance, the rate for the Tenino sandstone is from 1 to 2 feet per hour, while in the serpentine, which is a much softer material, at the United States quarry the rate is not more than from 4 to 6 inches per hour.

"The kind of power used for driving these saws varies, and may be steam, electricity, or water-power, and in Washington all three are used. Steam, however, is the one most commonly used, but is much more expensive than water-power.

"Machines of various kinds for planing and dressing marbles have been constructed and they are said to work very satisfactorily, producing a surface equal to a sand-rubbed finish and saving much labor and expense in the finishing of marbles.

"Up to the present time, however, nothing of this kind is being used in this State.

"Lathes of various kinds and sizes are in use in the mills for turning marbles and serpentines. The lathes are used for turning out columns of marble and vases and ornaments of various kinds from marbles and serpentines. These lathes are practically the same as those used for the turning of wood and iron. After the desired shape has been obtained and while the column or whatever it be is still in the lathe the polisher is brought into use and it is finished before it is removed from the lathe.

"For the rubbing of the stone smooth, preparatory to polishing, the most common contrivance is perhaps what is known as the rubbing-bed. This consists of a heavy cast-iron plate which revolves in a horizontal plane. The plate is revolved by means of a perpendicular shaft through the center, which is geared to the power, this gearing in some cases being above the bed, while in others it may be below. Above the revolving bed are a number of fixed arms, extending from the center to the outside of the bed and just high enough above it so as not to touch it. The slabs and blocks of marble are placed on this rubbing-bed, and these arms prevent their revolving with it when it is put in motion. Onto this plate are then put sand

or some other abrasive, and water. A large number of pieces may be placed on one bed at the same time and rubbed at much less expense than they could be by hand."

As to the general design of piers, very little further need be said than what has been given in the preceding chapter with reference to the cross-section that produces the least resistance in a stream, and the quotation from Morison as to the form of pier adopted in the various large bridges with which he was connected.

After the loads to be carried by the pier and the weight of the pier itself have been fully determined, the size of the base of the pier can be easily determined so as to keep the pressure upon the foundation bed inside of proper limits. The unbalanced pressure, due to wind and current, must be taken account of, that from wind being calculated by the ordinary methods of moments, while the force of the current may be calculated from the discussion given in Weisbach's " Mechanics." (See Chapter XXV.)

Should a particularly large base be required, it will be necessary to offset the courses of the pier by putting in several steps, and these offsets may be calculated by the ordinary formula for transverse loading of beams. Where the pressure on the foundation bed per square foot amounts to two tons, Baker gives the following coefficients by which to multiply the thickness of the masonry course to get the offset: for granite 1.5, for limestone and sandstone 1.3, and concrete in the proportion of 1, 3, and 5, 0.3. As this is the most usual unit pressure, this will be a sufficient guide in ordinary cases. In large piers where caissons and cribs are used, they are made of the proper size to properly distribute the bearing upon the foundation bed.

CHAPTER XXIV

ROCK FILL FOUNDATIONS AND QUARRIES

The use of riprap in connection with foundations is of such wide application that the engineer should have a very definite idea as to its production and use. The matter contained in this chapter is not to be taken as a treatise on rock excavation or quarrying, but simply the author's experience in operating quarries, and using riprap in river and harbor work, supplemented by various examples from the work of other engineers.

Riprap rock is used in making rock fills for railroad embankments along the water, for riprapping such embankments, for forming river dykes, for riprapping around bridge piers and wharves, and for the foundations of seawalls and breakwaters.

The quarry Figs. 332 and 333 is a hill about 150 feet high on the shore of Puget Sound, and is composed mainly of a dark gray volcanic tuff weighing 135 pounds to the cubic foot, and being somewhat friable, it is easily drilled and readily broken up into riprap sizes from 25 pounds to several tons in weight. The rock hardens after quarrying and especially in salt water.

The method of working the quarry, as may be seen from the illustrations, is of two distinct types, one portion of the quarry, Fig. 332, being worked on a bench about 20 feet above the water with guy derricks and gravity cars for handling the rock, while the other portion, Fig. 333, is worked from the water level by means of derrick scows, the cost of operating the latter being less than one-half the cost of that portion on the bench. For example, taking corresponding minimum costs, the rock handled by the floating derricks costs approximately: For drilling and blasting, 5 cents per cubic yard; mucking, 6 cents per cubic yard; breaking up and loading, 14 cents per cubic yard, or a total of 25 cents per cubic yard; while in the other portion drilling and blasting costs approximately 8 cents per cubic yard; mucking, 18 cents per cubic yard; and breaking up and loading, 35 cents per cubic yard, or a total of 56 cents per cubic yard. In both instances the coal, powder, and incidentals

Fig. 332.—Riprap Quarry on Puget Sound. No. 1.

Fig. 333.—Riprap Quarry on Puget Sound. No. 2.

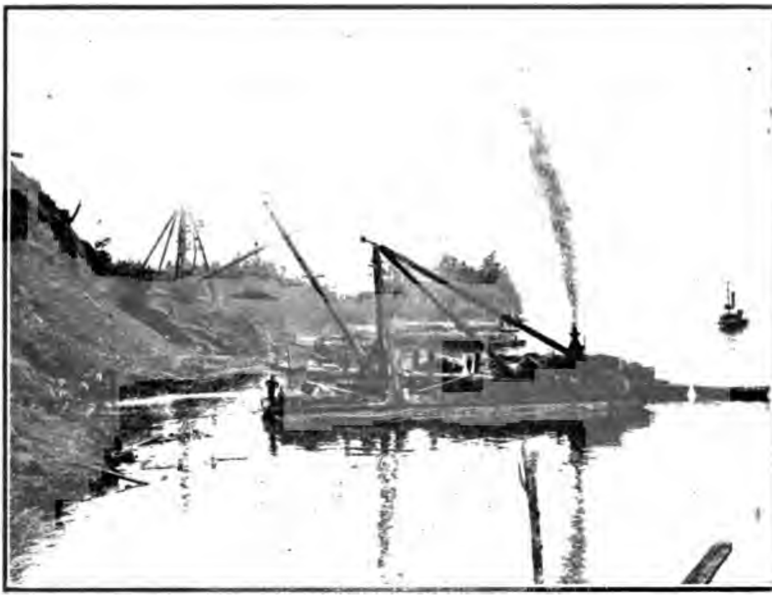


FIG. 333a.—QUARRY NO. 2, SHOWING DERRICK SCOW.

FIG. 333b.—QUARRY NO. 2 DERRICK SCOW LOADING BARGE.

average about the same, or 12 cents per cubic yard in addition to the above. Quarry No. 1 is operated by drilling deep top holes from 16 to 30 feet in depth with churn drills, and after springing with dynamite, these holes are fired with black powder, each shot bringing down from 3 to 7 yards of rock per pound of powder. In taking out the bottom bench along the face, air burleys are used to put in lifting shots, thus cleaning out the toe as the work progresses. The medium-sized rock is broken up by use of steel gads (Fig. 334) and sledges, and the larger pieces are drilled with air-plug drills and broken up with small charges of dynamite ranging from one-fourth of a stick to a whole stick for each hole. The rock is then loaded onto skips, which are swung around onto cars by means of a 3-drum steam engine and 10-ton derricks. The cars are operated on a double track, the loaded car pulling the light one back by means of a cable

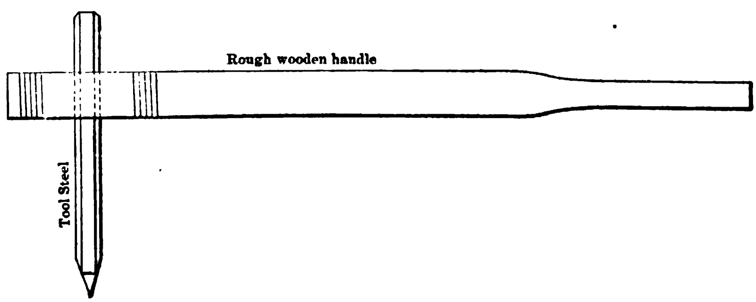


Fig. 334.—Rock Breaking Gad.

running around sheaves and over a brake drum. The distance to the dump is approximately 200 feet, and the rock is dumped on to scows which are afloat at high tide, and which rest on a grillage of timber or a "gridiron" at low tide.

Quarry No. 2 is operated much the same way as regards the shooting and breaking up of the rock, except that larger rock can be handled by the floating derricks directly from the quarry onto the scows, this being limited only by the size rock that can be readily unloaded and placed in the railway embankment.

Quarry No. 1 requires about 75 men to load two 225-yard scows per day, and quarry No. 2 requires only 50 men for loading two 300-yard scows per day, or a total output of the whole quarry of about 1000 to 1100 yards per day of ten hours. The wages paid for common labor was $2.75 per day, and regular rates for the higher class of men, or an average of about $3 per day.

The rock is loaded on the scows by first dumping from the cars, then piling it up into a rough wall around the outside to a height of 3 feet or more, leaving a walk-way around the sides and ends of the scow, after which the center portion is filled up to an approximate level, this making it easy to measure up the yardage on the scows without much error. Where rock is measured by the ton, tables are made for each scow, showing the tonnage for each inch of displacement, and by taking the draft of the scow on each side at both ends the number of tons of rock can be readily taken from the table of that particular scow.

FIG. 335.—PLACING ROCK ON SEAWALL, PUGET SOUND.

The scows when loaded are taken off the gridirons in No. 1 quarry, and off the bench in No. 2 quarry, and hauled to a deep water anchorage by the gasoline tug Fig. 263, and towed thirty miles by two tugs of the type illustrated in Fig. 270, each tug taking from two to three scows at a trip, and returning with from two to five empty scows, a round trip distance of about thirty miles; the rock being used to make a railroad seawall for a length of about twelve miles on the shore of Puget Sound, Fig. 335, and requiring for the entire work upwards of 300,000 cubic yards scow measure. The cost of towing and the cost of scows amounts to about 10 cents per yard for each

item. Three stiff-leg derricks on shore, and one floating derrick, of about 10 tons capacity each, are used in unloading and placing the rock to a rough one to one slope, which is carried on at a cost of approximately 20 cents per cubic yard. This cost and the various preceding ones are the results under the best conditions, where the run of the quarry is being loaded. Where bad weather and other troubles delay the work and where the rock has to be sorted or hand piled to slope, the costs given will be increased from 30 to 50 per cent.

The rock at the quarry was partially shot down by tunnels or coyotes, some of which were driven in to a distance of 150 or 160 feet from the face, and cross-headings made as shown in Fig. 336 (*a*) and (*b*), and loaded with black powder and dynamite, dynamite being used wherever the tunnels were wet, and about 4 yards of rock per pound of powder shot down. The data given in Rankine's Civil Engineering as to the amount of rock loosened per pound of powder agrees very closely with the author's experience. "The proportion of the *weight of rock loosened to the weight of powder exploded* ranges from about 7000:1 to 14,000:1 and may be taken on an average at 10,000:1." The dirt on top of the quarry, running from a foot or so to up to about 6 feet in depth, had been removed by teams and scrapers dumping most of it into a gully at one side of the quarry after the ground had been cleared and grubbed. Some of the dirt was trapped into a chute carrying it to the bottom of the quarry, and washed away with a sluicing plant.

This sluicing plant consisted of a $12 \times 7 \times 10$ duplex pump operated from two locomotive type boilers with a total of 90 horse-power; the main discharge pipe from the pump being 6 inches in diameter, with 4-inch branches to each side of the quarry, operating 4-inch hydraulic giants. These giants, Fig. 337, are easily handled by one man and at least one of them was required to be kept in constant operation (Fig. 338) in order to keep the quarry free of what little dirt was still left on top of the quarry, the dirt coming from the seams of the rock, and the fine débris from the breaking up of the rock. The dirt and débris in No. 2 quarry was very much less, and was largely washed away by the tide, and the balance easily dumped into deep water or to one side with the scow derricks.

The hand drilling was carried on by two men at each drill, making from 13 to 16 feet per day, each man receiving $2.75 per day. The holes were sprung with dynamite and loaded with from two to four kegs of black powder, several holes being shot off with a battery at one time. The block holes for breaking up the large rock were all fired

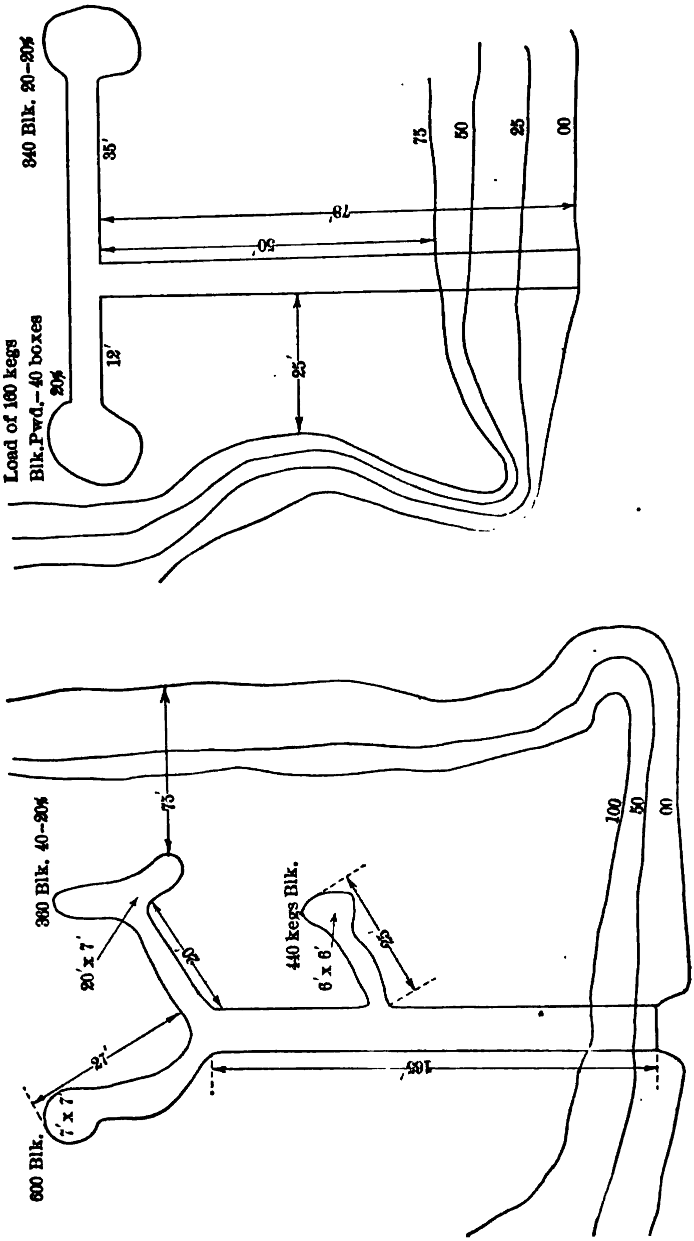


Fig. 336.—Coyotes in Riprap Quarry.

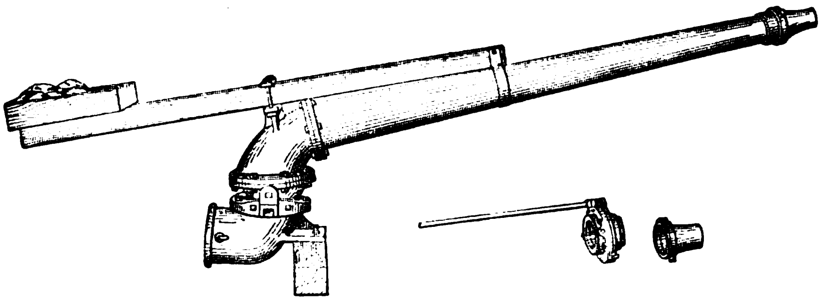


FIG. 337.—GIANT USED IN QUARRY SLUICING.

FIG. 338.—SLUICING OUT MUCK IN RIPRAP QUARRY.

with fuse. These holes were drilled with compressed-air plug-drills, Fig. 339, which are easily operated by one man. The burley drills, Table XLVI, were also operated by compressed air, and were of the Sullivan type, Fig. 340, and supplied with air from a Sullivan compressor, Fig. 341. This compressor was a 9×10×12, being sufficiently large to operate one burley and two of the plug-drills at the same time. This compressor is shown in Table XLV, but ordinarily, it would be better to have the compressor the 9×18×12, supplying

FIG. 339.—PLUG DRILLS FOR BLOCK-HOLING.

565 cubic feet of free air per minute for similar work, or large enough to operate three 3¼-inch burleys at the same time. The amount of air required for operating burley drills is shown in Table XLVII.

For the benefit of those desiring to purchase their first rock-drill plant, the following list comprises a complete outfit, which contains everything necessary for operation aside from boiler or air-compressor:

One rock drill, with throttle valve, oiler, wrenches, and extra set of packing for front head.

One universal joint tripod, with weights and wrench, ... sets of drill steels, sharpened ready for use to a depth of............. feet.

One............foot length of.........inch.............ply (marline or wire)............wound (steam or air)............... with connections.

One set of blacksmiths' tools (five pieces) for forming and sharpening the drill-steel bits.

One sand-pump for cleaning out the holes.

FIG. 340.—BURLEY OR TRIPOD AIR DRILL.

If for quarry work and quarry bar is required, substitute:

One steel quarry bar with weights and wrenches.

If for mining or tunneling and shaft bar or column is required, substitute:

One column with clamp.

The printed advice accompanying the Sullivan tools in regard to sharpening drills is worth repeating. "Too much emphasis cannot be placed on the importance of using suitable steel with rock

drills. The bits must be properly formed, sharpened, and tempered for the work in hand, or the drill cannot operate to advantage.

" It will prove economical to secure the best blacksmith obtainable to care for the drill steel. If bits of the right temper, shape, and sharpness are always on hand, the drills will be able to work constantly to the best advantage, whereas delays to the drills mean losses in efficiency to the whole plant.

" For general mining and quarrying purposes the ordinary cross-bit is recommended. The proportions of the bit, as to length and thickness of the wings or ribs, are indicated in the accompanying illustrations, Figs. 342 *a* to *d*. Figs. 342 *a* and *b* are bits for hard, non-gritty rock, and are alike except for the different angles shown

FIG. 341.—SULLIVAN AIR COMPRESSOR.

on the cutting edges. Fig. 342 *a* shows about the highest angle to which the cutting edge can be made without danger of breaking. The angle shown in Fig. 342 *b* on the cutting edge is one of many which may be used under different conditions without any other change in the bit. In cutting hard and medium hard rock, sharp drills and a wide-open throttle may be used to good advantage, and the hole will not ordinarily clog with mud, as the amount of rock loosened by each blow is so little that it is at once mixed into slush by the water in the hole. The sharp rebound of the drill when striking hard rock, together with the positive recovery of the machine, quickly gets rid of this slush. If the same bits and drill are run on an open throttle in soft or even medium soft ground, the hole soon becomes clogged. The reason for this is that, while the hole remains of the same diameter, and the amount of water for mudding purposes

TABLE XLV.—SULLIVAN STRAIGHT-LINE STEAM-DRIVEN AIR-COMPRESSORS.

Class.	Steam Cylinder. Inches.	Air Cylinder. Inches.	Stroke.	Piston Displacement Cubic Ft. of Free Air. Per Revolution.	Piston Displacement Cubic Ft. of Free Air. Per Minute.	Revolutions per Minute.	Maximum Air Pressure with 90 Lbs. Steam. Lbs.	Indicated Horse-Power in Steam Cylinder at Maximum Pressure.	Dimensions. Length. Ft.	Dimensions. Length. In.	Dimensions. Width. Ft.	Dimensions. Width. In.	Dimensions. Height. Ft.	Dimensions. Height. In.	Piping. Air Outlet.	Piping. Steam.	Piping. Exhaust.	Piping. Jacket.	Foundation Bolts.	Fly-wheel Diameter Inches.	Code Word.
WA3	8	8	10	.577	101	175	60–100	17–21	7	6	2	6	3	9	2	1½	2	½	¾	33	Kajac
WA3	9	9	12	.881	141	160	60–100	22–30	8	5	2	8	3	11	2½	2	2½	½	¾	36	Kajil
WA3	9	10	12	1.09	174	160	50–80	24–32	8	9	2	8	3	11	3	2	2½	½	¾	36	Kajon
WA3	9	12	12	1.57	251	160	30–50	19–34	8	9	2	8	3	11	3½	2	2½	½	¾	36	Kajpa
WA3	9	14	12	2.14	342	160	10–20	14–25	8	9	2	8	3	11	5	2	2½	*	¾	36	Kajro
WA3	9	18	12	3.53	565	160	10–15	23–35	8	9	2	8	3	11	7	2	2½	*	¾	36	Kajty
WA4	12	12	12	1.57	251	160	60–100	39–53	11	11	3	5	6	4	3½	3	4	½	¾	56	Kajut
WA4	12	14	12	2.14	342	160	40–75	41–61	11	11	3	5	6	4	4	3	4	½	¾	56	Kajvi
WA4	12	16	12	2.80	448	160	20–40	33–55	12	00	3	5	6	4	5	3	4	¾	¾	56	Kajwe
WA4	12	21	12	4.81	770	160	10–20	33–60	12	2	3	5	6	4	8	3	4	*	¾	56	Kajyas
WA4	12	24	12	6.28	1004	160	10–15	42–60	12	2	3	5	6	4	8	3	4	*	¾	56	Kajyk

* Without water jackets in air cylinders or cylinder heads.

TABLE XLVI.—PRICES, WEIGHTS, AND SPECIFICATIONS OF SULLIVAN ROCK DRILLS (UNMOUNTED).

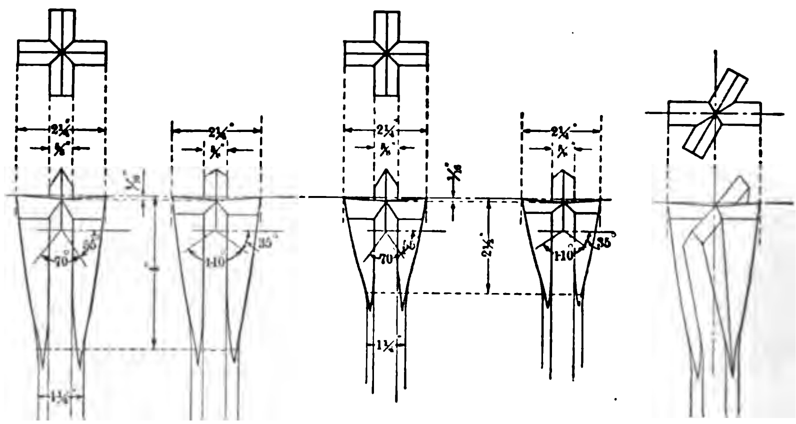
Letter Indicating Size.	UA	US	UB	UC, UC11	UD	UE2 UE11	UF2 UF11	UH UH11	UH2	UK	UL
Diameter of cylinder............in.	2	2¼	2½	2¾	3	3⅛	3¼	3⅝	3⅝	4¼	5
Length of stroke................in.	4½	5	5	6½	6½	6½	6¾	7¾	7½	8	8
Length of feed (depth drilled without changing steel)................in.	12	15	20	24	24	24	24	30	24	30	30
Depth of hole machine will drill easily ft. from 1 to	4	5	6	10	12	14	16	20	20	28	32
Diameter of holes that may be drilled in.	⅝ to 1¾	¾ to 2	1 to 2¼	1¼ to 2½	1¼ to 3	1½ to 3	1¾ to 3	1½ to 4	1½ to 4	2 to 5	3 to 6
Diameter of drill steel............in.	¾ to ⅞	⅞ to 1	⅞ to 1	1 to 1⅛	1⅛ to 1¼	1⅛ to 1¼	1⅛ to 1¼	1¼ to 1⅜	1¼ to 1⅜	1½ to 1⅝	1⅝ to 1¾
Number of pieces in set of steel to drill holes to depth above stated.......	4	4	4	5	6	7	8	8	10	10	13
Diameter of steam inlet..........in.	¾	¾	¾	1	1	1	1	1¼	1¼	1¼	1¼
Size of hose to connect to drill....in.	¾	¾	¾	1	1	1	1	1¼	1¼	1¼	1½
Size of steam pipe to carry steam 100 to 200 ft....................in.	¾	1	1	1	1¼	1¼	1¼	1½	1½	1½	1½
Size of boiler to supply steam for one drill.......................H.P.	5	6	8	8	8	10	10	12	12	15	15
Weight of drill unmounted.......lbs.	110	145	165	240	265	260	280	390	345	560	900
Shipping weight of drill boxed ...lbs.	145	180	200	280	310	305	325	475	420	775	1110
Price of drill unmounted...........	$200.00	$220.00	$240.00	$265.00	$300.00	$330.00	$350.00	$430.00	$430.00	$465.00	$500.00
Size of tripod....................	U1 or U2	U2	U3	U3	U3 or U6	U6	U6	U7	U7	U7	U9
Size of mining column or shaft bar...	U21	U21	U24	U24	U27	U27	U27	U29	U29	U29	U29
Code word for piston valve drill unmounted, for steam..............	Bajado	Bajanos	Bajesid	Bajillo	Bajith	Bajonula	Bajoujo	Bajular	Bajunac	Bajury	Bajuster
Code word for piston valve drill unmounted, for air................	Bajac	Bajame	Bajel	Baji	Bajoa	Bajoun	Bajuz	Bajuco	Bajust	Bajubi	Bajustice
Code word for tappet valve drill unmounted, for air................	....	....	....	Bajicar	....	Bajouthe	Bajuzzen	Bajuphe			
Code word for tappet drill unmounted, for stea	....	....	....	Bajilits	....	Bajouvey	Bajuzyf	Bajurad			

TABLE XLVII.—CUBIC FEET OF FREE AIR REQUIRED TO RUN FROM ONE TO FORTY ROCK DRILLS.

No. of Machines.	Amount Free Air per Minute. Rock Drills (Air Pressure, 75 Pounds per Square Inch at Ocean Level).									
	UA	US	UB	UC UCII	UD	UE2 UEII	UF2 UFII	UH, UHII UH2	UK	UL
	2 Inches.	2¼ Inches.	2½ Inches.	2¾ Inches.	3 Inches.	3⅛ Inches.	3¼ Inches.	3⅝ Inches.	4¼ Inches.	5 Inches.
1	55	70	84	100	112	123	135	149	180	214
2	110	140	168	200	224	246	270	208	360	428
3	149	189	227	270	303	332	365	403	486	578
4	187	238	286	340	381	419	460	506	612	728
5	226	287	344	410	460	505	554	611	739	878
6	264	336	404	480	538	590	648	715	864	1029
7	297	378	454	540	605	665	730	805	973	1157
8	330	420	504	600	672	738	810	894	1080	1284
9	358	455	546	650	728	800	878	968	1170	1392
10	385	490	588	700	784	861	945	1043	1260	1498
12	446	567	681	810	907	995	1095	1209	1460	1735
15	539	686	824	980	1100	1206	1324	1460	1765	2100
20	676	861	1035	1230	1378	1514	1660	1833	2215	2633
25	803	1022	1228	1460	1636	1796	1970	2176	2628	3125
30	935	1190	1430	1700	1904	2090	2295	2533	3060	3640
40	1190	1512	1815	2160	2420	2660	2920	3220	3890	4625

is therefore the same, the steel chips out three or four times as much dust at each blow as it does in hard rock. The rate of cutting should therefore be reduced in order to keep the drill working at maximum efficiency. The speed may be regulated by throttling the air or steam, but this reduces the rapidity of action of the drill, so that it does not always mix into slush the dust caused even at the slower speed. The recoil of the steel from soft rock is also considerably less. In soft rock duller bits should be used, like that shown in Fig. 342 (*d*). The angle of the cutting edge may be even higher than this, sometimes almost square on the end, in order to secure good results.

"In connection with the above subject, it is well to bear in mind the length of the wings or ribs for different kinds of work. Figs.



(*a*) (*b*) (*c*) (*d*) (*e*)

FIG. 342.—BITS FOR ROCK DRILLS.

342 (*a*) and (*b*) show an extreme length for very hard rock, intended to give strength and hold the gage as long as it is necessary. Figs. 342 (*c*) and (*d*) show shorter ribs, which give the bit more clearance and make it more desirable for general purposes. Under ordinary conditions its ability to mix mud is much greater than that of the long bit like Fig. 342 (*a*). For drilling dry holes in tunnel headings or elsewhere, the bit with short ribs had less tendency to allow the hole to draw up. The wings are five-eighths of an inch thick for the size shown in Figs. 342 (*a*) to (*d*), and should never be less than that for this size of bit and steel. They should be the same thickness throughout to allow free return of the cuttings. If gage less than $2\frac{1}{4}$ inches is desired, make the bit correspondingly shorter.

"In seamy ground, the bit shown in Fig. 342 (*e*) will occasionally

work satisfactorily when a cross-bit would not prevent a 'rifled' hole, since the '×' bit strikes only half as often as the '+' bit in a given spot. Flat or chisel bits are not recommended, since their reaming qualities are poor, and while cutting faster than the + bit under some conditions, they are very hard on the machine.

"It is important to keep the wings square at the corners, as this permits the gage of the hole to be properly maintained. Do not use a set of steel after the gage has begun to wear. The time and trouble taken in securing fresh steel amount to little in comparison

FIG. 343.—SKAGIT RIVER DYKES.

with the delay causing by trying to work down a hole with steel that is constantly sticking, to say nothing of the wear and tear on the machine. Blacksmiths should take pains to furnish bits made exactly to the required gage. A little neglect in this particular causes much trouble and loss of time to the drill.

"On any rock on which the cutting edges are not dulled upon the first hole, a system should be devised by the foreman or superintendent to determine how much each bit will do without too much 'hammer help.' The improvement will be very pronounced."

Where deep holes are to be drilled up to 35 to 45 feet in depth the Sullivan type "UL" drill must be used.

The compressor used was operated by a 60-horse-power boiler, and was supplied with a large receiver to insure steady pressure.

The equipment at the quarry in addition to the above comprised a full outfit of tools of all kinds, with a small machine shop and blacksmith shop for doing repair work. The camp for boarding the men was a model of its kind, having a large bunk house for the laborers, and a number of smaller houses for the skilled workmen. A large cook house and dining-room to feed 175 men at one time, and a reading-room, card-room, and store carrying all kinds of supplies were provided. The camp had its own sewerage system, water system, and shower baths for the men. The charge for board was $5.50 per week and regular prices were charged for supplies, resulting in fair profit.

The riprap placed around bridge piers is ordinarily from two-man stone down to small pieces of 5 or 10 pounds in weight, as the riprap will usually stand better when the voids are pretty well filled up. The seawall for the railroad embankment previously described required rock from 1 cubic foot in size upward, but the author's experience on work on Puget Sound indicates that a large amount of smaller rock should be used in order to fill up the voids, otherwise the wave action in stormy weather will wash out the sand bottom and cause the seawall to flatten out. This, of course, would be some different in swift water in rivers, or for heavy seas encountered in break-water and jetty work. In the first instance, the rock would have to be large enough to resist the force of the current, and in the second instance to resist the force of the waves and breakers. Several rock fills constructed by the author for protecting railway embankments were begun by following out the specifications excluding the small rock and quarry waste, but after finding that ordinary storms caused the flattening out of the rock work as before described, enough fine rock was used to completely fill the large voids, and no further trouble was experienced.

Many of the docks at Seattle extend out into water from 50 to 70 feet deep at low tide, and even by the liberal use of brace piles the piers are too limber and unsteady, and it is necessary to fill in around the piles up to within 35 or 40 feet below low water.

The work at Pier No. 6 on the Seattle waterfront for the Milwaukee Railroad was carried out by the author, by building a rock fill from 12 to 18 feet in height around the sides and end of the pier with riprap rock, and then filling inside with quarry waste and earth. Owing to the soft bottom the rock sank into the mud so that approximately 50 per cent more yardage was used than originally estimated.

This was done at a cost of $1.30 per cubic yard for the rock in place and 43 cents per cubic yard for the filling, the rock being rolled over the sides of the scow by hand, care being taken to prevent the scows capsizing by keeping them trimmed; and the earth was sluiced off by using a large hose attached to a city water main. Similar earth work for other piers was handled by using a large centrifugal pump, placed on a scow and operated by a steam plant.

The construction of about three miles of river dike, Fig. 343, on the Skagit River, Washington, carried out for the United States Government by the author, required about 20,000 tons of riprap rock running from two-man stone down to pieces of 30 pounds in weight. Two rows of piles were first driven as shown, and then several feet of brush fascines were placed on the sand and the rock placed on top of this. As rapidly as the work was completed it became buried in sand, and the channel deepened, following the work very closely. The cost of this work is shown in Chapter XXXII.

The use of riprap rock as the foundation of sea-walls is described in Chapter XXIX, where it has been extensively employed for support for seawalls in New York City. The use of riprap in the construction of the piers at South Brooklyn is described in Chapter XXVII.

The harbor work at Manila in the Philippine Islands required about 900,000 tons of riprap rock, which was towed across Manila Bay from Marivales, a distance of about thirty miles, by two large steel tugs, and a fleet of wooden scows about 30×125 feet in size. The rock was of moderately hard volcanic origin, and in the neighborhood of 300,000 tons were shot and broken up at one time nearly ready for loading, by the shooting of a large coyote. A cable-way running from the top of the hill to an anchorage in the bay made it possible to place the scows underneath it where they could be readily loaded by this means. The contract price was $2.50 per ton in place at sea-walls.

The work on San Luis Obispo Harbor, California, break-water was carried out under the following specifications:

The work to be done under these specifications will consist in furnishing, delivering and placing in position stone to repair and complete the breakwater now under course of construction at San Luis Obispo Harbor (Port Harford), California. So much stone as may be necesssary will be used in repairing the break-water already constructed and the remainder in extending the work.

The stone must be of suitable quality, not liable to disintegrate, generally angular in shape, and of random sizes, no piece to weigh less

than 10 pounds. The least dimension is not to be less than one-fourth the greatest, nor the weight per cubic foot less than 130 pounds when dry.

It is the intention to build the rubble mound approximately to a height of about 6 feet above mean high tide, with top width of 20 feet and with slopes on outer and inner sides of 1 on 2 and 1 on $1\frac{1}{2}$ respectively and an end slope of about 1 on 3. Any stone under 100 pounds must be confined to the lowest course of the work. Stone under 2 tons will be confined to the hearting of the mound below low-water depths of 6 feet, and must be covered by stone of 2 to 4 tons and upward up to low water. Above low water and as a covering for the slopes from low water to 10 feet below low water, stone from 4 to 8 tons in size will be used. In the top covering and in the covering for the slopes down to low water, stones only of 8 tons and upward in size will be used, except perhaps a small percentage of smaller stone for use in chinking between the larger. In building up the uncompleted wall and in the repair work, very little stone under 8 tons will be used. It is estimated from the present condition of the breakwater that about 40 per cent. of the stone may be in sizes under 2 tons, 30 per cent. in sizes of 2 to 4 tons, 20 per cent. in sizes from 4 to 8 tons and 10 per cent. in sizes of 8 tons and upward. Stone of the various sizes must be delivered as may be from time to time required and placed in the work where directed.

The use of large rock for the base of a breakwater in rebuilding jetties at Humboldt Bay, California, is described in "Professional Memoirs" by Morton L. Tower, Assistant Engineer; M. Am. Soc. of C.E.

Humboldt Bay, on the northern coast of California, latitude 40° 45′, is the principal lumber-shipping port of the State of California, being in the center of one of the most productive forest areas known to man.

The entrance to the harbor is between two low sand-spits. The area of the bay at low water is 11.4 square miles and at the contour 6 feet above low water it is 25.3 square miles. The mean rise and fall of the tide is 4.3 feet with extreme ranges as great as 11 feet, from $9\frac{1}{2}$ feet above to $1\frac{1}{2}$ feet below the plane of reference established by the United States Coast and Geodetic Survey at the mean of the lower low waters.

In its natural condition the erosion of the tidal currents, modified by the ever present but constantly changing surf conditions, resulted in a channel unstable as to position, depth, or direction, with a general ruling depth of about 12 feet at low water.

The harbor was first examined with a view to improving the condition of the entrance in 1877, but owing to the difficulties to be encountered and the then undeveloped stage of jetty construction methods, no recommendations for actual construction were made. The report stated that a safe entrance could be obtained by the construction of two parallel jetties about 500 yards apart.

Following an appropriation in 1881, control of the South Spit was attempted by the use of brush mattresses. The work being carried on with no appreciable result until 1887, when, following the development of construction methods at Galveston, Tex., and on the Oregon Coast at Yaquina Bay and at the mouth of the Columbia River, a plan for improving the channel conditions by means of two jetties of riprap stone deposited from pile trestles was adopted. Work on the jetties was commenced in 1887 and continued until 1899. A total of over 1,000,000 tons of stone was used, the cost of the work being $2,040,203.35.

The channel, both as regards alignment and depth, secured by the improvement was all that was expected and was maintained for several years at not less than 26 feet at mean lower low water with considerable periods when a ruling depth of 30 feet prevailed.

The jetties were constructed by depositing stone ranging in size up to pieces of 10 tons, from pile trestles—the method generally used on the North Pacific Coast.

The effect of the severe surf on these jetties has been to cause subsiding of the outer ends of the work, principally by reduction of slopes and by displacing the top stone. The smaller pieces of stone have been washed away and some disintegration of the stone has occurred. The mass has also settled into the bottom to some extent. Attrition by the sand-laden water is a source of possible loss, considering the total amount of surface exposed.

The desirability of using large-sized stone is a factor in jetty maintenance which has been well established by experience at all the North Pacific Coast harbors. In planning this work it was decided that the limiting size should be 20 tons. It was also considered desirable that these stones be lowered to place to avoid breaking them or the stone they fell on, which often occurs when stone is deposited by dumping from cars on an elevated track. The use of an unloading crane also permits the placing of stone in a selected position in the jetty, which cannot be done when it is dumped from a tramway.

All the quarries adjacent to Humboldt Bay are at a considerable distance from the navigable channels, thus rendering rail transportation essential from the quarry to tide water. The quarry used is 7 miles from the nearest landing.

The largest single item of plant involved in the construction is the cars. It was deemed that it would be cheaper in the end if

these were of standard design and hence salable when the work was completed.

A tramway of sufficient strength to carry a 20-ton crane and standard 40-ton railroad equipment is necessarily much heavier and more expensive than the jetty tramways used along the Oregon and Washington coasts, where narrow-gage, special dump-cars are used.

The storms of many years had beaten the old enrockment into a compact mass, such that it would have been impossible to drive piles into it, and for a tramway it would have been necessary to place a portion of the piles of each bent in the sand to give it any lateral stability.

Stone for the work is supplied under contract by the Hammon Construction Company at the following prices:

	Per ton.
Class 1	$1.74
Class 2	1.56
Class 3	1.50
Class 4	1.50

The following is printed from the specifications under which the stone is being supplied:

DESCRIPTIONS OF MATERIAL

Class 1 Stone. To be of large pieces only, weighing from 10 tons to 20 tons each piece. These stones will be used for slopes on the outer end of the work, and none will be received until the jetty repairs have been extended 2300 feet from the high-water shore lines. Delivery of this stone will be required up to 500 tons per day when work is in progress on the outer ends of both jetties. Stone of Class 1 will be loaded directly on the flat car without the use of skips.

Class 2 Stone. To be in pieces of 1000 pounds each to pieces of 10 tons each, in the following proportions:

One-fourth of each day's supply may be in pieces of 1000 pounds to pieces of 3 tons; one-half of each day's supply must be in pieces of from 3 tons to 6 tons each; and one-fourth of each day's supply must be in pieces from 6 to 10 tons each.

This class of stone will form the major portions of the repairs to the jetties. It is expected that the delivery required will not be less than 1000 tons per day after the work has been well commenced on both jetties, and about 500 tons per day for the first season's work.

Stones of Class 2, when in pieces under 6 tons each, must be loaded on suitable skips holding up to 10 tons of stone each. Skips will be strong and designed for lifting at the four corners. Four corner hooks will be provided on each skip for the convenient attachment of the unloading crane spider chain. Skips will be kept in good working order by the contractor.

Class 3 Stone. Will be used only for bringing the top of the rough mound to a nearly smooth tight surface, and for concrete displacers. It will be in pieces not less than 3 pounds nor more than 500 pounds each, delivered on skips similar

to those above described. Stone of Class 1 or Class 2 must not be loaded on cars carrying stone of Class 3. Stone of Class 3 will be used in decreasing amount as the jetties are extended. From 120 tons per day at the commencement of the work to about 10 tons per day when the outer end of the work is reached.

Class 4 Stone. Will be used for making concrete. It may be clean crusher run of the same rock as used for the other classes, or a good selected or washed river gravel may be used. It must vary in size from pieces with greatest dimension not more than 3 inches to the finest product of the crushers. If river gravel be used it must be thoroughly washed, all organic matter of any nature removed, and screened if necessary to exclude pieces greater than 3 inches largest dimension. Either gravel or stone must be of hard durable rock which will not disintegrate in the finished work.

Broken stone or gravel will be loaded on flat cars, provided with suitable sides. The amount required will vary from about 70 tons per day during the first part of the work to 8 tons per day for the outer ends of the jetties.

Stone of all classes must be hard, close grained, and not liable to disintegrate in the work. It must be free from seams of any nature and of uniform grain and texture. It must be delivered free from all dirt, quarry refuse, or foreign material of any nature, preferably in nearly cubical blocks. The greatest dimension of any piece will not exceed five times the least dimension. It must weigh not less than 150 pounds per cubic foot, dry. Bidders will submit samples of the stone they propose to deliver and will name the location of the quarry or quarries which they propose to operate. Stone will not be accepted for use in the jetty until the quarry from which it is to be supplied has been approved by the contracting officer. A compact stone of high specific gravity is desired. The weight per cubic foot and quality of the rock, as well as the price per ton, will be considered in making the award.

The United States reserves the right after at least a season's trial to require that no further deliveries of stone of Classes 3 and 4 be made, but that the amount of stone to make up the undelivered portions of the 30,000 tons and 14,000 tons mentioned in paragraph 18 be furnished of Class 1 or Class 2, or partly of each class.

The approval of a quarry by the contracting officer shall not prevent the rejection of any stone not complying with the requirements herein specified.

The larger portion of the stone supplied is a close-grained igneous rock, weighing about 198 pounds per cubic foot. It is very difficult to quarry, breaking into very uneven fragments. However, by proper manipulation, a minimum of waste is secured and, as there is no covering soil to be contended with, the quarry is very satisfactory.

A second stone supply from the same vicinity is a close-grained metamorphic sedimentary rock with irregular planes of division of argillaceous material. This stone weighs 167 pounds per cubic foot and is easily worked. On account of the seams and a considerable covering of soil, which cause a large amount of waste, it has not been found advantageous to furnish any considerable quantity of this stone so far.

The stone is loaded by the contractors on standard flat cars and hauled 7 miles to a loading point on a navigable channel, where it

is placed on barges carrying eight cars each. When delivered at the jetty receiving plant, it is unloaded and the empty cars are returned to the barges.. From the loading point to the jetty landing is 9 miles.

The above arrangement permitted the contract to be made for the material only and the contractors have nothing to do with the actual jetty construction. The United States is not required to pass on the rock until it is offered for use at the jetty wharf.

The contractor's crew has numbered generally about one hundred men, employed for seven ten-hour days per week. The following plant has been installed by the contractors:

Four 20-ton stiff-leg derricks, 100-foot booms, with steam-driven hoisting and swinging engines.

One steam crane, with shovel attachment, used for grading pits and tracks and for loading cars.

One two-stage, $16 \times 10 \times 14$-inch, Ingersoll-Rand cross-compound air compressor, electrically driven.

One small jaw rock-crushing plant, electrically driven.

In addition, there is the usual equipment of air-drills, small tools, shop and mess equipment and appliances.

Hollow drill bits are used, and since the installment of an air-driven Leyner sharpener no difficulty has been experienced in successfully quarrying the stone. The contractor's transportation plant consists of fifty flat cars, 60,000 pounds capacity, 36 feet long, two car ferry barges and two towboats. From the quarry to the landing the cars are handled over a logging railroad by the logging companies' motive power.

The contractors are now providing fifteen additional cars and a third barge.

The receiving and depositing plant at the jetty, belonging to the United States, consists of a 100-foot span, three-track apron for transfer from barge to shore tracks, adjusted to tidal elevation at barge end by counterweights, and fixed at 10-foot elevation at shore end; two locomotives; three flat cars for miscellaneous materials; a 20 cubic-foot concrete mixer mounted on a flat car; a 10-ton revolving and traveling unloading crane, gage of gantry 14 feet; water supply and distributing system; fuel oil storage and distributing system; a repair plant with power-driven tools for ordinary blacksmith, carpenter, and light machine work; an electric-light plant; store house, mess house; crew quarters and necessary minor equipment for the work in progress. A new stone-unloading crane of 20 tons capacity is now in course of construction.

The jetty crew varies from forty to fifty men working six eight hour shifts per week.

The method of construction is: The enrockment is first brought to an elevation averaging 2 feet below the finished grade with Class

2 stone; pieces weighing from 1000 pounds to 10 tons. None of the pieces of Class 2 stone are allowed to project above 6 inches below grade—the bottom of the ties. Voids in the mass are then filled with Class 3 stone, pieces weighing from 3 pounds to 500 pounds, and then top is leveled off at from 18 to 10 inches below grade. Holes are choked by hand-placed stone. A rough form is made by tying wale-pieces, 6×8 inches ×20-foot ties, together with wires 5 feet apart, and nailing to them short vertical boards with bottoms in contact with the rock. A rock dam is built at the front end of the form. Concrete is mixed rather dry, deposited with a cubic yard self-dumping and self-righting bucket, handled by the stone-unloading crane. The concrete is brought to within an inch or so of the bottom of the ties. The end tie is brought to grade, the crane rails laid, and the ties placed and spiked. Concrete is then continued to the top of the ties.

The concrete mixer is mounted on the end of a standard flat car, the discharge shoot delivering over the end. Mixing water is supplied by gravity from a tank on the opposite end of the car. Oil fuel is supplied by gravity from a tank near the water tank. The oil and water tank also supply the stone-unloading crane. Oil is pumped and water flows by gravity to the crane supply tanks. Cement for a day's operation is carried on the concrete mixer car.

The concrete is machine mixed in a Foote Batch Mixer of 21 cubic feet capacity, end-discharge type, steam driven.

CHAPTER XXV

CALCULATION OF PIERS, FOOTINGS AND RETAINING WALLS

THE calculation of the stability of piers and retaining walls, and the calculation of the strength of footings is not entirely germane to the subject of this treatise, but in order to properly design and execute the foundation work it is necessary for the engineer to be able to fix the dimensions for piers, retaining walls, and footings.

Where piers have been properly designed to distribute the load on the foundation, and have been given the ordinary batter of from ½ to 1 inch per foot, there is little chance, even with the highest piers that are likely to be constructed of masonry, of any lack of stability under any ordinary combination of forces, and yet the engineer should go over the calculations for stability just as carefully as if there were real danger of failure. For wide city bridges and double-track railroad spans the investigation of stability becomes quite perfunctory, but for high piers with long spans for highway work, for lift bridges, and for single-track railway spans on high piers, the calculations should be carried out with great care in order to be positive that conditions have been fully taken care of.

The forces acting upon and lengthwise of the pier (Fig. 344) and with the current tend first, to overturn the pier; second, to slide it upon some section between the coping and the base; third, to slide it upon the base; fourth, to crush it at some section of the pier; or, fifth, to cause the failure of the foundation bed itself.

The forces acting crosswise of the pier (Fig. 345) and at right angles with the current and in line with the bridge are first, the pressure of the wind on the side of the pier; second, the force caused by the expansion and contraction of the spans; and third, the skidding force of the train. Other forces may occur, as in the case of movable or bascule spans resting on piers; collision of boats; log jams, or the lodging of heavy drift.

Taking up the forces acting lengthwise of the pier, the first effect to be considered is that of the overturning moment. The elevation of the pier shown in Fig. 344 has the points of application of the

forces shown by the letters A to G. A and B are the points of application of the wind blowing against the bridge trusses. The greatest pressure of the wind (Table XLVIII) may be taken at 50 pounds per square foot on 1.8 times the exposed surface of one truss,

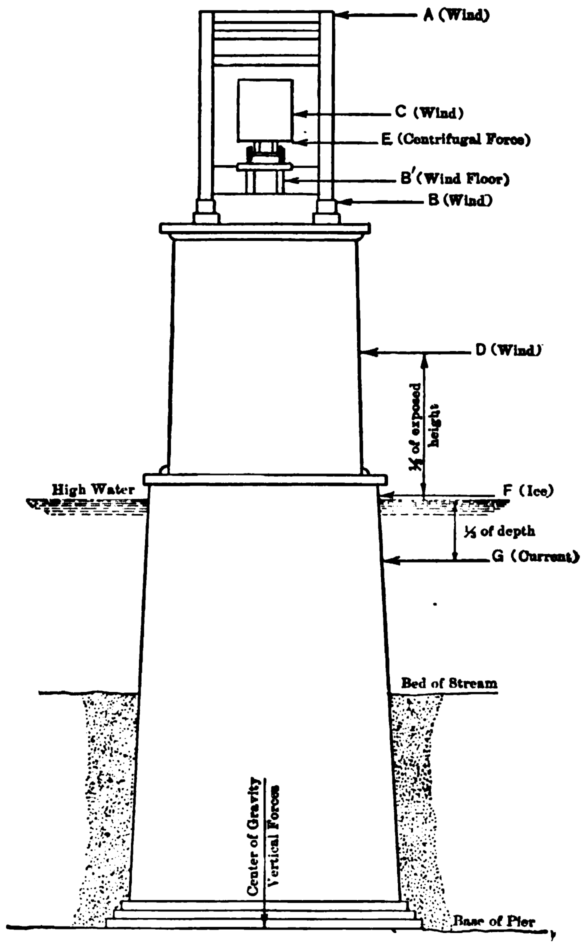


FIG. 344.—LONGITUDINAL PIER STRESSES.

or corresponding to a wind velocity of over 100 miles per hour. This pressure need not be considered as acting when a train is passing over the bridge, as it would be unlikely that a train would venture out on to a bridge in such a gale, and where the force is considered as coincident with the passage of a train it may be taken at 30 pounds

per square foot, corresponding to a wind velocity of about 80 miles per hour. These two forces comprising the total wind load on the trusses can be considered as acting half-way between *A* and *B*. In addition to these it will be necessary to figure the wind pressure on the floor system at the same pressure per square foot, acting at a point above *B*, at the center of the side elevation of the floor system.

TABLE XLVIII.—WIND PRESSURE.

Smeaton's Formula $P = .005 V^2$.

Miles per Hour.	Feet per Minute.	Feet per Second.	Force in Lbs. per Sq. Ft.	Description.
1	88	1.47	0.005	Zephyr
2	176	2.93	0.020	Very Light
3	264	4.40	0.045	
4	352	5.87	0.080	Light Breeze
5	440	7.33	0.125	
10	880	14.7	0.500	Ordinary Breeze
15	1320	22.0	1.125	
20	1760	29.3	2.000	Ordinary Gale
25	2200	36.6	3.125	
30	2640	44.0	4.500	Strong Wind
35	3080	51.3	6.125	
40	3520	58.6	8.000	Very Strong Wind
45	3960	66.0	10.125	
50	4400	73.3	12.500	Hard Storm
60	5280	88.0	18.000	Very Hard Storm
70	6160	102.7	24.500	
80	7040	117.3	32.000	Hurricane
90	7920	132.0	40.500	
100	8800	146.6	50.000	

The force *C* is the wind pressure on the exposed area of a train, and may be taken as acting at a height of 8 feet above the rail on a surface of 10 square feet per lineal foot of the train, or equal to 300 pounds per lineal foot of train, on the basis of 30 pounds per square foot. Complete calculations of stability should be made with and without the train.

The force *D*, or the wind pressure acting on the shaft of the pier, is very seldom considered, but in a strict analysis it should be taken into account and will, of course, be an amount obtained taking the width of a face of a rectangular pier, multiplied by the height above the water, multiplied by 30 or 50 pounds per square foot, as the case may be, and acting at one-half of the height from the surface of the water to the top of the pier. For a pier with rounded ends this force would only be about 0.5 of that upon the face of a rectangular pier.

The force E is the centrifugal force of the train acting at say 4 feet above the rail, where the track is on a curve. This force may be calculated from the formula:

$$F = .00001167\ V^2DW.$$

V = speed of train in miles per hour;
D = degree of curvature of track;
W = weight of the train coming upon the half spans resting upon the pier.

The force F is the pressure exerted by moving ice in rivers where ice is likely to run. This force is ordinarily figured at 300 pounds per square inch, but on the St. Louis bridge it was figured as high as 600 pounds per square inch on the area estimated to be covered by the crushing ice. Where ice or log jams occur the pressure will probably be equal to the hydrostatic pressure on the space occupied by the pier and the half span on each side, from the hydrostatic head due to the difference in the elevation of the water above and below the bridge. More than likely the force of an ice jam will be one entirely beyond the bounds of calculation, and the judgment of the engineer must govern the alllowance to be made for rivers in very cold climates; although it will probably not be necessary to go to the great extreme of building such heavy piers with ice breakers, as were constructed by Robert Stephenson for the Victoria bridge over the St. Lawrence River. The force of the ice is to be taken as acting at the level of high water.

The force G is the pressure due to the current, which may be considered as acting at one-third the depth below high water. This pressure in pounds per square foot may be taken at a value determined by Weisbach's formula:

$$P = WK\frac{v^2}{2g}.$$

W = weight of water per cubic foot = 62.43 pounds;
v = velocity in feet per second;
g = acceleration of gravity = 32.2 feet per sec. per sec.
K = 1.47 for square piers;
K = 1.33 for rectangular piers;
K = 0.73 for cylinders;
K = 0.67 for piers pointed with arcs of a circle;
K = 0.60 for elliptical piers.

The value obtained from this formula is based on the maximum velocity of the stream, and while the average velocity is very often taken at one-half the maximum, it is well to be on the safe side and use at least two-thirds of the value as obtained from the formula.

These forces acting with their respective lever arms, equal to the distance from the point of application above the base of the pier, or the section under investigation, are offset by the moment of the vertical weights, first, of the spans; second, the weight of the train (when the wind force is taken at 30 pounds per square foot); third, the weight of the pier, less the buoyancy of the water if it is possible for the water to get under the base of the pier, taken at 62.43 pounds per cubic foot, for the number of cubic feet of the pier immersed in water. Should the support afforded by the side friction of the pier be taken into account, this also should be deducted from the weight of the pier; but, on the other hand, if this is taken account of, the resistance of the ground from the bed of the stream to the base of the pier acting to prevent overturning must also be taken account of, at such a value as judgment dictates from a study of the ultimate bearing value of the soil, very small in silt and reasonably large in packed sand or gravel; so that in making an investigation of a pier's stability, a section at the bed of the stream or a small distance below, should be investigated as well as at the base of the pier.

The moment of these vertical forces is usually calculated by taking a lever arm equal to the distance from the resultant line of their action to the leeward boundary of the pier, but this lever arm should never be used, but one used of such a length to some point within the pier as judgment and careful investigation would dictate. Where the piers are founded on piles, or upon a yielding bottom, a reduced lever arm must be used, taken from the center of application of the vertical forces to the center of gravity of an area next the end of the pier, where the maximum load possible to be carried by the piles or the material would act. Where the pier is designed with a factor of safety of eight, this would reduce the lever arm by one-sixteenth the length of the pier.

The forces acting on the piers transversely, or in the direction of the center line of the bridge (Fig. 345) will consist of the forces X, Y, and Z; X being the skidding force or 0.2 times the total weight of train on the adjoining half spans.

The expansion force Y is equal to the weight of the half span expanding on a pier, multiplied by the coefficient of friction, running from 0.1 to 0.3; this will not act if large well-acting rollers are used,

but will come into play if the rollers are too small, or are rusted so they will not act.

The force *Z* is the wind pressure on the surface of the pier taken at 30 pounds per square foot with a train on the bridge, or 50 pounds

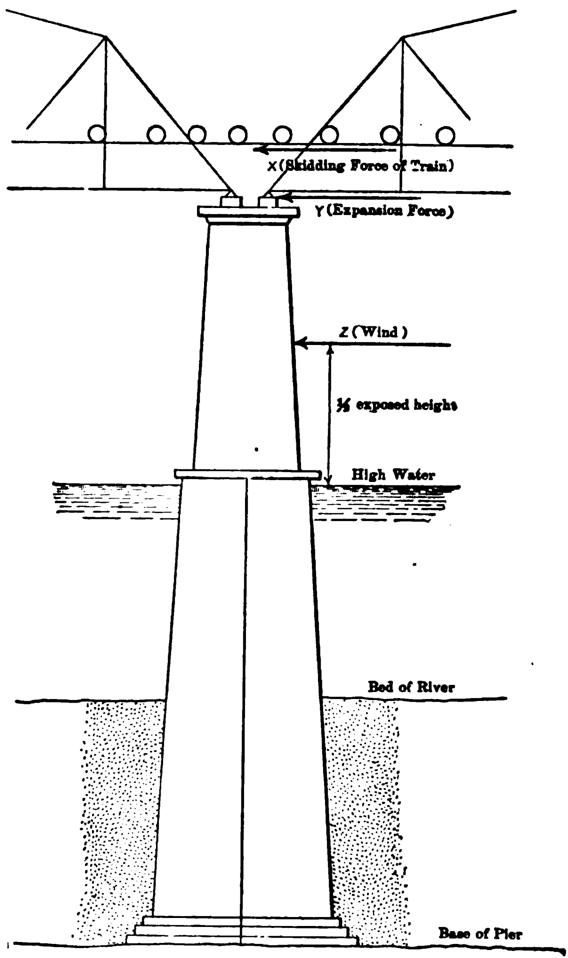


FIG. 345.—TRANSVERSE PIER STRESSES.

per square foot without the train; but will not be considered to act to cause sliding if full wind pressure is figured longitudinally. The moment of these forces will be obtained with the lever arm of *X* and *Y*, from the points of application of the forces as shown, to the section being investigated, or to the maximum at the base of the pier.

The force Z is to be considered as acting at the center of the exposed area above the water; the transverse resisting moment of the pier is to be obtained in exactly a similar manner to that already described for the longitudinal forces.

The resultant of the forces in both directions tending to overturn the pier, will act to slide the pier upon any section under investigation, or upon the base of the pier, and the resistance to sliding on the base will never be less than 0.33 times the weight of the spans, train, and pier as previously determined for masonry on moist clay, up to a maximum value 0.75 of the total weight so obtained, for masonry with slightly moist mortar, at any section above the base.

The investigation into the direct crushing of the pier at any section can be carried out by determining the total load above the point to be investigated, and comparing that to the crushing value of the material of which the pier is composed. The failure of the foundation would usually only come about by the total weight on the foundation determined as above, exceeding the maximum load the foundation would carry; it should usually have a factor of safety of eight, although it should never have a less factor of safety than four against failure. When it is deemed necessary for a high pier to investigate the compression caused on the leeward side of a pier by the forces acting on it, this can be found by using the method given in Baker's "Masonry Construction," tenth edition, page 353. The pier is then considered as a vertical cantilever beam fixed at the bottom and the moment of the horizontal forces computed; then the

$$\text{Total unit pressure at the leeward side} = P = \frac{W}{S} + \frac{Ml}{2I};$$

$$\text{Total unit pressure at the windward side} = P = \frac{W}{S} - \frac{Ml}{2I};$$

W = Total vertical load on pier;
S = Area of horizontal cross-section;
M = Moment of horizontal forces;
l = Length of pier;
I = Moment of inertia of section about a gravity axis perpendicular to the horizontal forces;
I = For rectangle $= \frac{bl^3}{12}$;
b = Thickness of pier.

The allowable pressure on masonry of various kinds is given in Table XLIX, and the allowable safe load on foundations as compiled by the author is given in Table L, as supplementary to those given in Chapter XXI. The calculation of the offset for footing courses has been discussed in Chapter XXI, and the most valuable of the data given in the University of Illinois Bulletin No. 67 on reinforced footings is quoted verbatim at some length, but for additional information as to the calculation of reinforced footings the reader is referred to the various other bulletins mentioned in the following pages, and to the various treatises on Reinforced Concrete.

TABLE XLIX.—SAFE LOAD FOR MATERIALS IN PIERS.

Short Tons per Square Foot. Factor 8.

Material.	T. Sq. Ft. (a)	Material	T. Sq. Ft.
Granite	90	Sandstone (ord.)	50
Basalt	85	Sandstone (good)	65
Marble	75	Rubble (ord.)	3
Limestone (good)	60	Rubble (good)	5
Oolites (good)	20	Portland concrete 1 : 5	15
Brownstone (bridge)	65	Portland concrete 1 : 8	12
Brickwork (ordinary)	3	Portland concrete 1 : 10	9
Brickwork (in cement)	4	Coignet beton	15

TABLE L.—SAFE LOAD ON FOUNDATIONS.

Usual Maximum in Short Tons per Square Foot when Material is Confined.

Material.	T. Sq. Ft. (a)	Material.	T. Sq. Ft.
Fine sand	4.5	Loose beds with piling	2.0
Sand and gravel	5.0	Loose beds with concrete	3.5
Sand and clay	5.0	Brick, stock mortar	3.0
Moist sand and clay	1.5	Brick in cement	4.0
Alluvium and silt	1.5	Rock, very soft	5.0
Hard clay	5.0	Rock (hard as concrete)	10.0
Firm stone on dry clay	4.0	Rock (moderately hard)	25.0
Hardpan	8.5	Rock (very hard)	45.0

The values given for bearing power of various foundation beds in column one of Table L are the usual maximum values to be used at a depth of about 23 feet below the bed of the stream, thus making a condition of actual confinement that prevents spreading. As the foundation bed approaches the bed of the stream from this depth, the allowable reduced pressure may be obtained from the following formula by the author:

$$p = 0.68a + 0.014ah$$

p = allowable pressure tons sq.ft.
a = constant value Table L.
h = depth below bed of stream.

Should the conditions be peculiarly good at greater depths than 23 feet below the bed of the stream, the values derived from the formula may be used up to a maximum of $1.4a$.

The subject of reinforced concrete wall footings and column footings is covered in the University of Illinois Bulletin No. 67, by Prof. A. N. Talbot. The following is taken verbatim from that paper:

Footings form an important element in the design of masonry structures. The two forms of footing most commonly used may be named the wall footing and the column footing, the former projecting laterally on the two sides of a longitudinal wall and the latter extending in four directions from the base of a column or pedestal block. It is usually assumed in the design of foundations that the earth conditions are such as to make the upward pressure on the footing uniform over its surface. Wide differences exist in methods of designing, due to differences in the assumptions made with reference to the structural action of the footing. It is not strange that these differences exist, since little or no experimental data are available which apply directly to the conditions of footings. Relatively short and deep beams and slabs under heavy uniform loads, with the supporting pressure largely concentrated at the center of the structure, may not be expected to give the same results as have been obtained in tests with the more slender beams and slabs and with the methods of support and of application of load which have generally been used in tests. With the present extensive application of reinforced concrete to footings, especially in connection with tall buildings carrying very heavy column loads, a more definite knowledge of the structural action of footings has come to be of importance. It is appreciated that the tests herein described are applicable only to a limited field, but they are offered as a contribution on a subject in which little experimentation has been done.

It may seem strange, considering the wide variations in practice, that few failures of footings have been publicly reported. It must be remembered, however, that these structures are out of sight, buried deep in the earth without opportunity for inspection. A failure in a footing may effect a change in the distribution of the load over the bed of the footing, resulting only in increased settlement. Possibly many instances of undue settlement of buildings may be due to failure in the footings. Possibly, in other cases, the earth at the center of the footing may be able to take the increased load under the conditions of side restraint developed. It is also probable that many footings have been made unduly strong.

The tests of 114 wall footings and 83 column footings are described in the bulletin. The wall footings were 12 inches wide, generally 5 feet in length and 12 inches in depth or 10 inches to the center

of the reinforcing bars, with a 12×12×12-inch stem in the middle to represent the wall through which the test load was applied. The wall footing rested on a bed of springs arranged in such a way as to approximate conditions of uniform upward pressure on the bottom surface of the footing. A variety in method of reinforcement was employed to throw light on the development of tensile stress in the steel and on the resistance to bond, diagonal tension, and shear. Tests of brick footings, unreinforced concrete footings, and footings having I-beams encased in concrete were included in the investigation of wall footings. The column footings were 5 feet square and generally 12 in. thick or 10 in. to the center of the reinforcing bars, and had a 12×12×12-in. pier built over the middle through which the load was applied. The column footings also were tested on a bed of springs which gave conditions approximating those of uniform upward pressure. Variety was given to the amount and method of reinforcement and to other conditions with a view of determining the structural action with respect to tension, bond, diagonal tension, and shear, and to give information which would bear upon methods of calculation of stresses. It is thought that these are the first experimental tests on column footings, and probably the first on wall footings on a bed of springs. Analyses are given of the stresses in wall footings and column footings and methods of calculation are discussed and compared with the results of the tests.

4. *General Theory.*—In wall footings and pier footings the weight or load is applied vertically through the wall or base block or pier, and the upward bearing pressure of the soil (which may also be called the load, since its amount and distribution determine the stresses) supports this weight from below. The usual assumption on which design of footings is based is that the soil pressure is uniform over the bed of the footing. Before uniformity of pressure on the footing will obtain, the footing must bend to the amount and form which would be caused by a uniformly distributed load. The assumption of uniform pressure is warranted if the earth layer is an elastic compressible soil of considerable thickness and of not too high a modulus of compressibility, as under these conditions the amount of bend of the projection of the footing is slight in comparison with the amount of compression of the earth. Also, in soft soils which flow laterally, as in a so-called floating foundation, the settlement and changes in the soil will produce conditions approximating uniform pressure. Where the bed is rock the pressure will be transmitted more nearly directly from the wall or pier to the rock, and as the projections of the footing have little opportunity

for being bent upward this portion of the footing may be expected to take only a small part of the load. This lack of uniformity of distribution of pressure is more likely to be present with reinforced footings than with the less flexible unreinforced footings which would carry the same load.

The principles of beam action are, in general, applicable to wall footings, but not so fully to column footings, which partake more of the nature of slabs. The formulas for calculating stresses in reinforced concrete beams have been treated in Bulletin No. 4, "Tests of Reinforced Concrete Beams: Series of 1905," and in Bulletin No. 29, "Tests of Reinforced Concrete Beams: Resistance to Web Stresses." The principal formulas for beam action in rec-

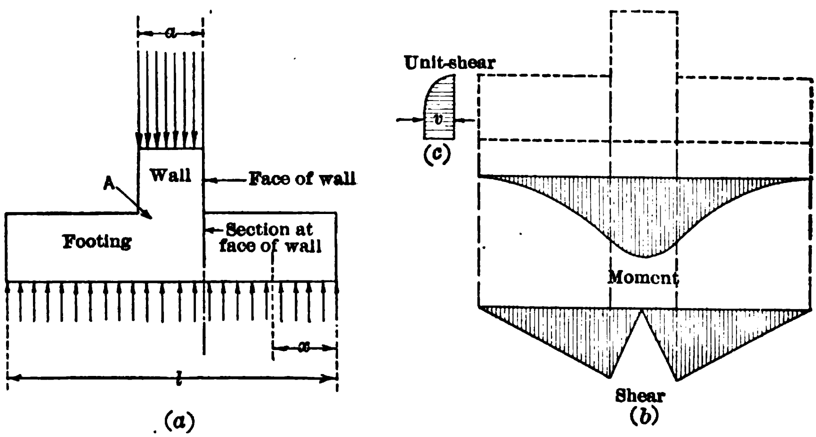


FIG. 346.—DISTRIBUTION OF LOAD AND PRESSURE IN FOOTINGS.

tangular beams reinforced for tension only, as used in this bulletin, will be repeated here.

The resisting moment of the reinforced concrete beam is (see Bulletin No. 29, page 6)

$$M = Afd' = Afjd, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

where A is the area if cross-section of longitudinal reinforcement, d is the distance from the compression face to the center of the longitudinal reinforcement, d' is the distance from the center of the reinforcement to the center of gravity of compressive stresses, j is the ratio of d' to d (which, for the beams of this bulletin, may be considered to vary from .82 to .92), and f is the tensile stress per unit of area in the metal reinforcement.

The formula for the maximum vertical shearing unit-stress in the concrete in any vertical section is

$$v=\frac{V}{jbd}=\frac{V}{bd''}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (18)$$

where V is the total vertical shear at the given section (equivalent to the resultant of vertical forces on one side of the section considered), and b is the breadth of the beam. This formula neglects any horizontal tensile stresses in the concrete.

The formula for bond unit-stress in horizontal reinforcing bars is

$$u=\frac{V}{mod''}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (17)$$

where o is the periphery of one longitudinal reinforcing bar, m is the number of bars, and the other symbols are as used before. This formula neglects any horizontal tensile stresses in the concrete.

These formulas were derived for certain assumed conditions in the beam. Since it is convenient to use them as a means of comparison for conditions other than those assumed, as, for example, when the bars are bent up at the end, the values obtained from these formulas will sometimes be referred to as nominal vertical shearing stresses and nominal bond stresses.

The value of the maximum diagonal tensile unit-stress in any section when tensile stresses exist is

$$t=\frac{1}{2}s+\sqrt{\frac{1}{4}s^2+v^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (19)$$

where s is the horizontal tensile unit-stress existing in the concrete and v is the horizontal or vertical shearing unit-stress. The direction and amount of this maximum diagonal tensile stress will vary with the relative values of s and v. In general, it may be said that in the ordinary reinforced concrete beam the value of t probably varies from one to two times v. This applies to the parts where tensile stresses exist in the concrete. Where the tensile strength of the concrete has been exceeded, it is customary to use the same formula.

It is evident that the value of the diagonal tension is generally indeterminate. No working formulas are available. For this reason it is the practice, now becoming nearly universal, in beams

without web reinforcement to calculate the value of the vertical shearing unit-stress v, and to use this as the measure or means of comparison of the diagonal tensile stress developed in the beam; with the understanding, of course, that the actual diagonal tension is considerably greater than the vertical shearing stress. It has been found that the value of v developed in beams will vary with the amount of reinforcement, with the relative length of the beam, and with other factors which affect the stiffness of the beam.

The following summary of the experiments gives the conclusions.

Wall Footings.—The tests of wall footings cover a variety of reinforcement. The method used to secure a distributed upward pressure introduced difficulties in testing. It also made it difficult to determine the load which should be taken as the critical load, and the loads which have been so specified may not always be the true critical load. The use of the bed of springs on the whole proved very satisfactory and is probably the best available arrangement for tests of the number and range used. The tests bring out phenomena which might not be apparent from analytical considerations alone or which might not be accepted without physical verification. Variations in concrete add to the complications encountered in analyzing such a series of tests. The tables and diagrams and discussions present information and data of the tests in a detailed way. The following statements summarize in a general way some of the points which are brought out by the tests and which have a bearing upon the principles and methods of design:

1. Wall footings under load follow the general laws of flexure. The section for maximum moment, the critical section for calculation of vertical shearing stress for use in judging of resistance to diagonal tension, and the method of calculating bond stress received experimental consideration.

2. The values of the modulus of rupture found in the unreinforced concrete footings are not far from the values of modulus of rupture obtained in simple beam tests such as the control beams. Increasing the richness of the mixture gives the added strength which tests of simple beams would lead us to expect. Variations in the tensile strength of concrete are to be expected, and considerable variation was found in the moduli of rupture of the test pieces, the variation being augmented by differences incident to the method of testing. The tests on footings of different lengths, undertaken to determine whether the section at the face of the wall should be used for the critical section, do not disclose any marked differences in modulus of rupture.

3. The results of the tests and the measurements of deformation of the reinforcement indicate that the critical section may be considered to be at the face of the wall and that the calculated tensile stress in the bars at this section is probably somewhat above the maximum tensile stress developed. Whether the maximum compressive stress may properly be calculated in the same way was not determined. It may be expected that high compressive stresses exist at the intersection of wall and projection. Indications of high compression and of incipient compression failure were found at the intersection of the wall and footing at loads above the critical loads.

Test pieces in which the wall was poured after the footing had taken its set, gave results which indicate that a section at the face of wall may properly be used in calculations of moments even when the wall is to be poured separately from the footing.

4. The calculations for bond stress, based upon the total external vertical shear at the section at the face of the wall and calculated by Eq. (17), evidently give stresses higher than the existing stresses. This is shown by the fact that the values calculated in this way are higher than those found in pull-out tests and beam tests. A study of the analytical conditions existing at this section tends to confirm the statement. However, as bond resistance is so important a strength element in a short cantilever beam, this method of calculation and the use of the working value of bond stress ordinarily assumed in design seems only reasonably conservative and may be recommended for general practice. Attention may properly be called to the importance of making calculations of bond stress in wall footings and other beams in which the length is short relatively to the depth. The advantage of using relatively small bars in such cases is also apparent.

Anchorage of bars by bending upward and back in a long curve or by looping the bar in a horizontal plane was found to add materially to bond resistance.

5. The tests indicate that the vertical shearing stresses developed at the face of the wall, calculated by the usual method, are higher than the vertical shearing stress which is found to exist in simple beams with concentrated loading when diagonal tension failures are developed. It was found that diagonal tension failures start at a point some distance away from the section at the face of the wall. This observation and certain analytical considerations such as the probable greater proportion of shear taken in the compression area at sections near the face of the wall show that, in calculating the vertical shearing stress which shall be used as a basis for judging the resistance to diagonal tension, a section some distance from the face of the wall

should be used. The tests and the discussion indicate that a section *d* distant from the face of the wall (*d* being the distance from the center of reinforcing bar to top of footing) may properly be used as the critical section for calculating the vertical shearing stress for this purpose, and that at this section the ordinarily accepted working stress may properly be used for calculating resistance to diagonal tension failure.

6. The bending up of bars at several points along the length of the projection gave added resistance against diagonal tension failure. Vertical stirrups also added to the resistance against diagonal tension failure but were not especially effective. Neither method of web reinforcement would be very convenient in construction. Generally speaking, it will be best to try to design the footing so that the vertical shearing stresses will be within the limit of the working stress permitted in beams without web reinforcement, and thus avoid the use of web reinforcement. In large important footings, when diagonal tension is a critical element, it would seem that some kind of unit frame with well-formed web reinforcement would be preferable to placing stirrups or to bending up bars at the necessary intervals. In stepped and sloping footings attention should be called to the larger diagonal tension and bond stresses developed. The increase in these stresses over those found in footings of uniform depth may be sufficient to decide against the use of stepped and sloping footings.

7. The footings having I-beams embedded in the concrete carried high loads, perhaps corresponding to the yield-point tensile strength of the lower flange of the I-beams and more than double what would be carried by naked I-beams. The weight of the I-beams, of course, was greater than that of the reinforcing bars used in the reinforced concrete wall footing.

Column Footings.—The requirement of uniform load and the presence of double-curved flexure complicate an investigation of column footings. In this investigation methods of testing were developed. As these are presumably the first tests on column footings, the phenomena of the tests and data of their action will be of interest to designers, especially in the directions in which tests have brought out weaknesses not always recognized and usually not guarded against. The results contribute data toward the settlement of methods of calculating of both the bending moment and the resisting moment for square footings, and the principles may with care be extended to other forms. The results may not easily be summarized, but the following statements are intended to cover the principal matters brought out in the tests:

1. A square column footing under load may be expected to take a bowl-shaped form. In slabs subject to bending in two directions, the stress in a fiber cannot differ from that in an adjoining fiber at the same level without setting up longitudinal shear; and as there is considerable resistance to variation from equality of stress in adjoining fibers, it may be expected that if stiff thick pieces (as are footings of ordinary design, where the thickness is large in comparison with the length of the projection) the deformations and consequent stresses will be distributed over the width of a cross-section and that considerable stress will be developed even in the fibers at the edge of the footing.

2. For footings having projections of ordinary dimensions, the critical section for the bending moment for one direction (which in two-way reinforced concrete footings is to be resisted by one set of bars) may be taken to be at a vertical section passing through the face of the pier. In calculating this moment, all the upward load on the rectangle lying between a face of the pier and the edge of the footing is considered to act at a center of pressure located at a point half-way out from the pier, and half of the upward load on the two corner squares is considered to act at a center of pressure located at a point six-tenths of the width of the projection from the given section. By equating this bending moment and the resisting moment which is available at the given section, the maximum tensile stress in the concrete or in the reinforcing bars may be calculated.

3. As is usually the case when plain concrete is used in flexure, the unreinforced footings show considerable variation in results. The variations were such as not to permit a method of determining the effective width of resisting section to be established or to obtain a formula for resisting moment. Based upon the full section of the footing, the moduli of rupture obtained were considerably less than the moduli of rupture of control beams made with the same concrete.

4. In reinforced concrete column footings, resistance to non-uniformity of stress in adjoining bars will be given by bond and by longitudinal shear in the concrete, and the amount of variation from uniformity of stress in the various bars will depend upon the spacing of the bars as well as upon the relative dimensions of the footing. With two-way reinforcement evenly spaced over the footing, it seems that the tensile stress is approximately the same in bars lying within a space somewhat greater than the width of the pier and that there is also considerable stress in the bars which lie near the edges of the footing. For intermediate bars stresses intermediate in amount will be developed. For footings having two-way reinforcement

spaced uniformly over the footing, the method proposed for determining the maximum tensile stress in the reinforcing bars, is to use in the calculation of resisting moment at a section at the face of the pier the area of all the bars which lie within a width of footing equal to the width of pier plus twice the thickness of footing, plus half the remaining distance on each side to the edge of the footing. This method gives results in keeping with the results of tests. When the spacing through the middle of the width of the footing is closer, or even when the bars are concentrated in the middle portion, the same method may be applied without serious error. Enough reinforcement should be placed in the outer portion to prevent the concentration of tension cracks in the concrete and to provide for other distribution stress.

5. The method proposed for calculating maximum bond stress in column footings having two-way reinforcement evenly spaced, or spaced as noted in the preceding paragraph, is to use the ordinary bond stress formula, and to consider the circumference of all the bars which were used in the calculation of tensile stress, and to take for the external shear that amount of upward pressure or load which was used in the calculation of the bending moment at the given section.

An important conclusion of the tests is that bond resistance is one of the most important features of strength of column footings, and probably much more important than has been appreciated by the average designer. The calculations of bond stress in footings of ordinary dimensions where large reinforcing bars are used show that the bond stress may be the governing element of strength. The tests show that in multiple-way reinforcement a special phenomenon affects the problem and that lower bond resistance may be found in footings than in beams. Longitudinal cracks form under and along the reinforcing bar due to the stretch in the reinforcing bars which extend in another direction, and these cracks act to reduce the bond resistance. The development of these cracks along the reinforcing bars must be expected in service under high tensile stresses, and low working bond stresses should be selected. An advantage will be found in placing under the bars a thickness of concrete of 2 inches, or better 3 inches, for footings of the size ordinarily used in buildings.

Difficulty may be found in providing the necessary bond resistance, and this points to an advantage in the use of bars of small size, even if they must be closely spaced. Generally speaking, bars of $\frac{3}{4}$-inch size or smaller will be found to serve the purpose of footings of usual

dimensions. The use of large bars, because of ease in placing, leads to the construction of footings which are insecure in bond resistance. In the tests the column footings which were reinforced with deformed bars developed high bond resistance. Curving the bar upward and backward at the end increased the bond resistance, but this form is awkward in construction. Reinforcement formed by bending long bars in a series of horizontal loops covering the whole footing gave a footing with high bond resistance.

6. As a means of measuring resistance to diagonal tension failure, the vertical shearing stress calculated by using the vertical sections formed upon the square which lies at a distance from the face of the pier equal to the depth of the footing was used. This calculation gives values of the shearing stress, for the footings which failed by diagonal tension, which agree fairly closely with the values which have been obtained in tests of simple beams. The formula used in this calculation is $v = \frac{V}{bjd}$, where V is the total vertical shear at this section taken to be equal to the upward pressure on the area of the footing outside of the section considered; b is the total distance around the four sides of the section, and jd is the distance from the center of reinforcing bars to the center of the compressive stresses. This stress is somewhat larger than the average vertical shear over the section which is sometimes used. The working stress now frequently specified for this purpose in the design of beams, 40 pounds per square inch, for 1 : 2 : 4 concrete, may be applied to the design of footings.

The punching shear may be calculated for the vertical sections which inclose the pier footing, although it may be expected that shear failure may not be produced exactly on this section. The value now generally accepted for punching shear, 120 pounds per square inch for 1 : 2 : 4 concrete, may be used for the working stress in this case.

7. No failures of concrete in compression were observed, and none would be expected with the low percentages of reinforcement used. The compressive stresses in the pier of the footing were in some cases very high and in a few instances the pier failed and was replaced by a cube of concrete. In frequent cases there were signs of distress near the intersection of pier and footing where there is an abrupt change in direction of surfaces and where the combined stresses are very high.

8. In stepped footings, the abrupt change in the value of the arm of the resisting moment at the point where the depth of footing changes may be expected to produce a correspondingly abrupt increase

of stress in the reinforcing bars. Where the step is large in comparison with the projection, the bond stress must become abnormally large. It is evident that the distribution of bond stress is quite different from that in a footing of uniform thickness. The sloped footing also gives a distribution of stress which is different from that in a footing of uniform thickness. However, for footings of uniform thickness the bond stress is a maximum at the section at the face of the pier; in a sloped footing the bond stress at the section at the face of the pier would be less accordingly than in a footing of uniform thickness, and a moderate slope may be found to distribute the bond stress more uniformly throughout the length of the bar. This is not of advantage if the full embedment of the bar is effective in resisting any pull due to bond.

9. The use of short bars placed with their ends staggered increases the tendency to fail by bond and cannot be considered as acceptable practice in footings of ordinary proportions. In footings in which the projection is short in comparison with the depth the objection is very great.

10. Footings having reinforcement placed in the direction of the diagonals as well as parallel to the sides (four-way reinforcement) gave good tests. The significance of the results is so obscured by the variety of manner of failure (bond, diagonal tension, and perhaps tension) and by variations in the quality of the concrete, that a comparison with two-way reinforcement on the basis of loads carried would not be of value. This type of distribution of reinforcement should be included in further tests. Measurements of deformation in the bars are needed to determine the division of stress among the four sets of bars.

Concluding Remarks.—The tests of wall footings and column footings leave uncertainty in some parts of the problem and there are gaps in other parts. The recent development of the portable extensometer or strain gage and the skill and experience which have been gained in its use in recent tests have opened opportunities for obtaining information on the stresses developed in such test pieces which were not available when the series of tests was undertaken. It is suggested that some of the remaining unsolved problems may most readily be attacked by measurement of deformations in the steel and concrete, and that further investigation may best be carried on by constructing a form of apparatus which will permit such measurements to be taken under the conditions of uniform loading.

The calculation of retaining walls has been the subject of numerous investigations, resulting in various formulæ, but for the purposes of

this treatise the method of calculation as given in the Engineer's Year Book has been quoted in full.

Let AB, Fig. 347, be the surface of the ground, and OB the natural slope at an angle ϕ.

Draw OE, making an angle θ, this angle being equal to ϕ plus the angle of friction of earth against the back, AO, of the wall $AOKL$.

Take X so that $EA \times EB = (EX)^2$.

Then AOX is the triangle of maximum pressure against the wall.

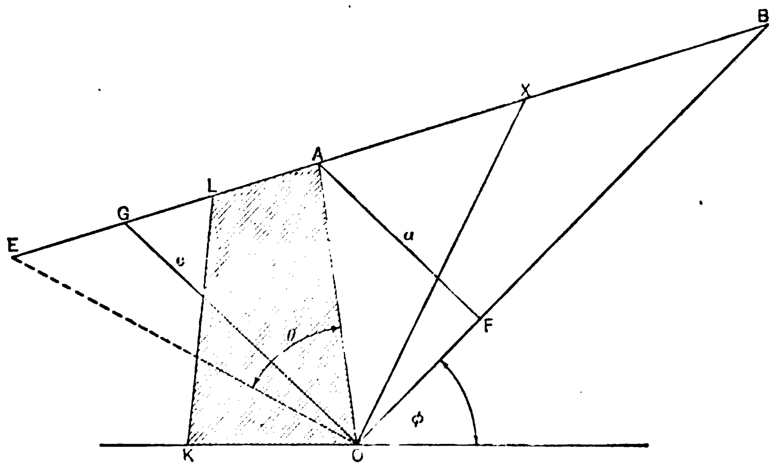


FIG. 347.—RETAINING WALL DIAGRAM.

Draw AF (a) and GO (c) perpendicular to OB, then,

$$\text{Maximum thrust} = \tfrac{1}{2}w(c - \sqrt{c(c-a)})^2,$$

where w = weight of a cubic foot of earth.

Describe a semicircle on GO (Figs. 347 and 348). Draw AD parallel to OB, cutting the semicircle at D. With G as center and GD as radius describe a circle cutting GO at P, and join DP, then,

Maximum thrust $= \dfrac{w}{2}(OP)^2$, acting at $\frac{1}{3}$ AO, and making an angle θ with the normal to AO.

Walls for Railway Cuttings

For walls over 18 feet in height: thickness at bottom $= \frac{2}{3}$ height -3 feet.

For walls less than 18 feet high: thickness at bottom $= \frac{1}{3}$ height $+3$ feet.

The two equations give the same result for a height of 18 feet. No allowance is necessary for a surcharge of less than 6 feet.

(Pennsylvania Railroad Co.)

Pressure of Water

The moment of pressure of water on a retaining wall per each foot of length of wall is $d^3 \times 10.4$, where d is the depth of water in feet. This is the moment at 1 foot. The total pressure is $31.2d^2$, assumed to be concentrated at one-third the depth from the bottom of the water. Hence the moment $10.4d^3$. The weight of the wall may be assumed concentrated at its center of gravity, and the effect

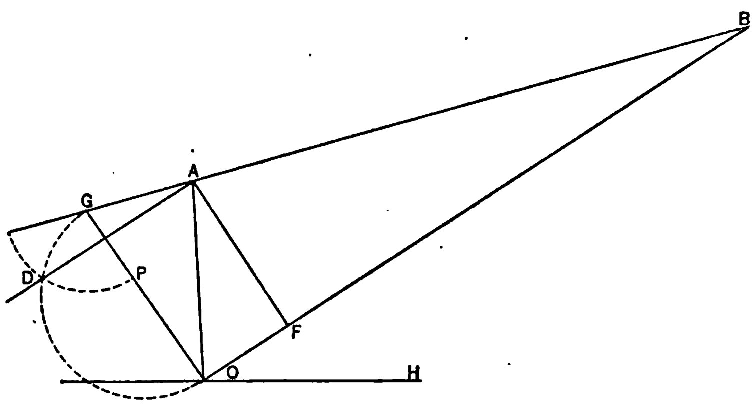


FIG. 348.—RETAINING WALL DIAGRAM.

of the pressure of the water is to turn the wall upon the point O (Fig. 349). Calling H the height of the wall in feet, and t its thickness in inches, and assuming 1 cubic foot to weigh 120 pounds, the moment of stability is $\frac{Ht^2}{2.4}$ for rectangular walls, per foot of length; or $d^3 \times 10.4$ must not exceed $\frac{Ht^2}{2.4}$; or $t^2 = \frac{25d^3}{H}$. This enables the thickness of a wall to be determined, that will exactly balance the pressure. Safety would be just secured by adding 50 per cent. to the thickness at the base, and subtracting the same amount from the top thickness, thus giving the wall a sloping back as per dotted line, increasing the acting leverage of the CG (center of gravity) of the section, so that the resultant pressure will now fall within the middle third of the wall thickness. This method is approximate and useful

for small construction. Large masonry dams require more special consideration, and may be designed by calculations at each few feet of depth, designing the wall in a series of steps and drawing a final shape to enclose the whole in one harmonious section, which must also fulfill the conditions of strength proper to the material and crushing stress. Special care is necessary that water does not get in between bed joints, as it would greatly assist to turn a wall over.

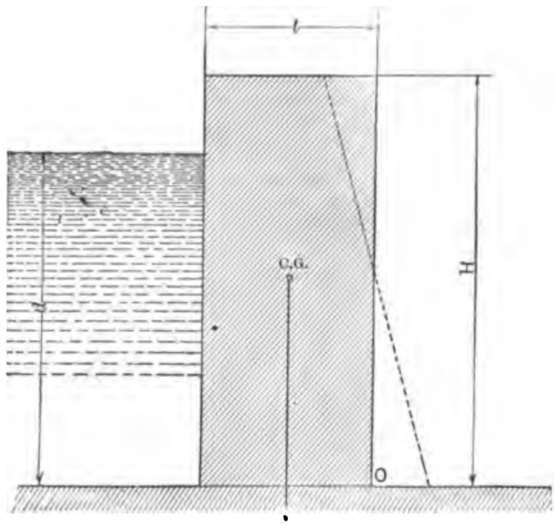


FIG. 349.—PRESSURE OF WATER ON WALL.

It is usually considered desirable that the resultant of pressure on a wall should fall within the middle third of its thickness. Thus in Fig. 350 if *ab* to any scale represents the total pressure $31.2d^2$ concentrated at the center of pressure upon the face of a wall, and *bc* to the same scale is the weight of the wall, the line *bc* being drawn through the *CG* of the cross-section, then if the diagonal of the rectangle *abcd* cuts the base of the wall within its middle third of thickness, the wall may be assumed fully safe. If the diagonal fall outside the middle third, the thickness of the wall must be increased. It may be made safe by building the wall higher, and thus loading it; but the danger of this method is that a greater area is exposed to wind pressure, and if margins of safety are narrow the addition may be worse than useless in small work. Where a tank wall forms

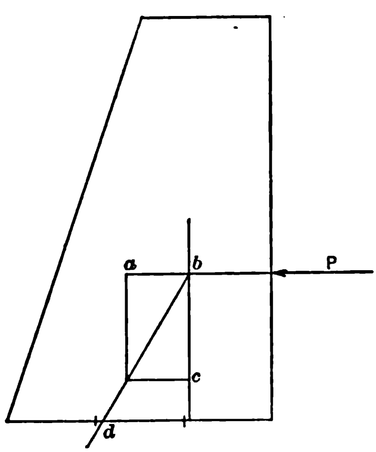


FIG. 350.—LOCATION OF RESULTANT OF PRESSURE.

a continuation of an earth retaining wall, for example, special care is necessary.

The upper part of the wall must be safe at the bed joint level with the bottom of the tank and the wall, taken as a vertical cantilever subject to a lateral thrust at a distance two-thirds of the water depth from the water surface; it must be safe against overturning at any joint lower down. It must also be safe against the added thrust of the earth behind the lower portion plus the further pressure due to the tank full of water above, which must be treated as per the rule of page 508, Fig. 353. Thus in Fig. 351 the wall must be safe on the line A against the moment of the water pressure Pc. It must be safe at the line B against the moment $(P\times d)+$ the moment $(E\times f)+$ the moment $(S\times n)$. Here E is the horizontal component of the earth pressure, and f is one-third AB; S is the horizontal component of the surcharged load, and n is one-half of AB. The safety must be proved at sufficient other depths between B and A and below B. Economy is promoted by sloping the wall against the earth between the points A and B. The wall in the figure is palpably erroneous, being far too thin for its height; it might be made safe by a sufficiency of cross-ties buried in the concrete bottom of the tank, in which case the wall from A to B must be made safe by treating it as a beam exposed to a virtual load at S and E.

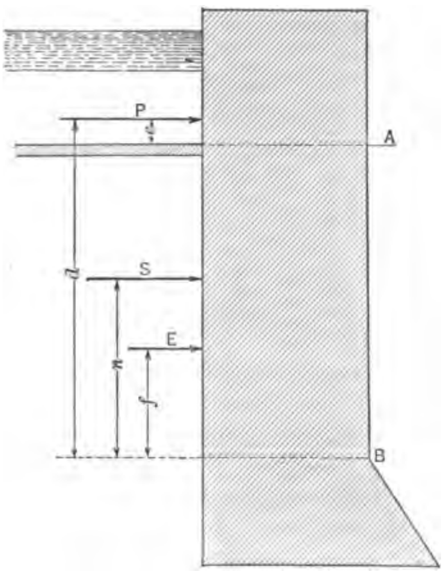


FIG. 351.—WATER PRESSURE ON RETAINING WALL.

Equilibrium of Retaining Walls

By Rankine's theory the pressure of earth on a vertical plane, AB (Fig. 352), may be considered as acting at a point, P, one-third AB from base A. and in a direction parallel to the surface. Let

w = weight of earth in pounds per cubic foot, ϕ = angle of repose of earth, P = total pressure, h = height of wall in feet.

For a bank with horizontal top

$$P=\frac{wh^2}{2}\cdot\frac{1-\sin\phi}{1+\sin\phi}.$$

For bank with any surface slope, θ, of indefinite length

$$P=\frac{wh^2}{2}\cos\theta\,\frac{\cos\theta-\sqrt{\cos^2\theta-\cos^2\phi}}{\cos\theta+\sqrt{\cos^2\theta-\cos^2\phi}}.$$

For bank with maximum surface slope, ϕ, of indefinite length

$$P=\frac{wh^2}{2}\cos\phi.$$

For bank with surface slope of definite length an intermediate value is taken.

To resist the pressure there is weight of wall and of earth, *ABD*, resting on its back, which call W; this acts vertically from center of gravity, G. The revetment may fail by overturning round f, by crushing at f, by sliding on Af.

Overturning Round f.—On any scale make $oa=W$, $ob=P$; then oc is resultant (R), acting in direction oe at e. If e falls within base, the revetment will not overturn without crushing the toe at f; but e should fall within the center third of Af.

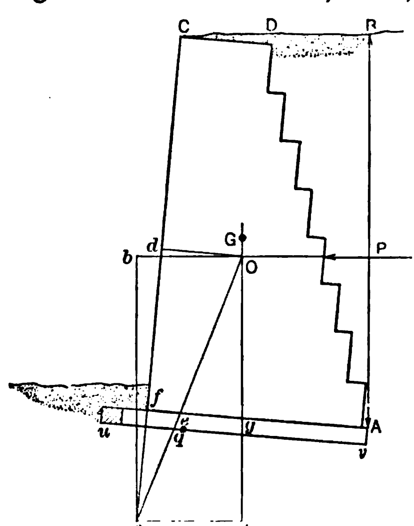


FIG. 352.—EQUILIBRIUM OF RETAINING WALL.

Crushing at f.—The resultant, R, must be resolved into two forces, cd (N), perpendicular to, and do parallel to, Af, acting at point e; the former is the crushing force.

Pressure per square inch at f on $1'$ length of wall $= \dfrac{2N}{fg \times 12''}$, where $fg = 3fe$.

The pressure at f should not exceed power of material to resist crushing, divided by factor of safety, 4 to 8 (in practice seldom as high as 8).

Sliding on Af.—Force tending to produce sliding is od; force tending to resist it is $N \times$coefficient of friction at the joint Af; od should not exceed $N \times \frac{4}{5}$ coefficient of friction.

Surcharged Wall

If n = height of wall in feet, c = height of surcharge in feet, t = mean thickness of wall in feet, to sustain horizontal bank; t' = mean thickness of wall in feet, to sustain bank with indefinitely long natural slope, the factor of safety being about the same as for t; t'' = mean thickness in feet of surcharged wall; then

$$t'' = \frac{nt + 2ct'}{n + 2c}.$$

Effect of Weight of Buildings on Retaining Walls

Let w_1 = weight of building per unit of surface; the rest of the notation as at commencement. The total hroizontal pressure, P, on revetment, due to the weight of building

$$= w_1(h-x)\,\frac{1-\sin\phi}{1+\sin\phi},$$

acting at $\frac{1}{2}OB$ from O; and the total horizontal pressure, P_1, on revetment, caused by the earth,

$$= \frac{w(h-x)^2}{2}\cdot\ \frac{1-\sin\phi}{1+\sin\phi},$$

acting at $\frac{1}{3}OB$ from O.

If the front of the building were farther back from AO the effect would be less, but cannot be determined.

Foundations of Retaining Walls

Find the pressure perpendicular to *uv* (Fig. 352), the weights and pressures being taken from point *v* instead of from *A*. If *q* is center of pressure, *qu* should nearly equal *qv*, in order that the pressure may be uniformly distributed (this is unnecessary if the pressure at *u* is within what the earth can bear without yielding), and greatest pressure at *u* should not be greater than earth will bear, usually 1 to $1\frac{1}{2}$ ton, or say 2500 to 3500 pounds per square foot. Foundations to be carried deep enough to avoid effects of heat and frost.

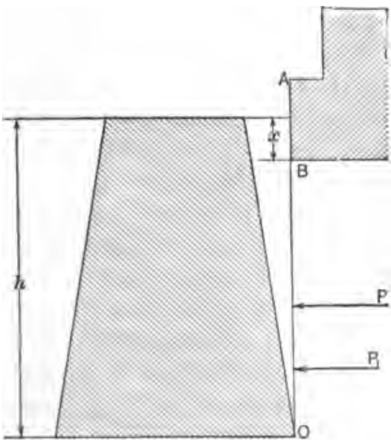


FIG. 353.—SURCHARGED RETAINING WALLS.

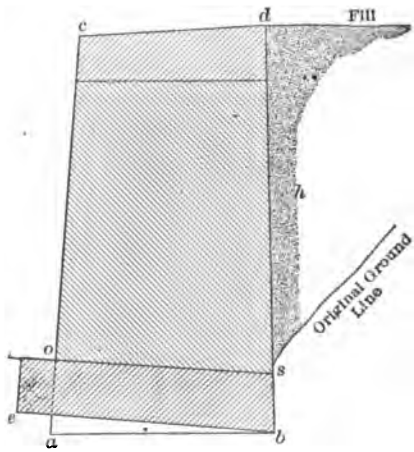


FIG. 354.—RETAINING WALL, TRAUTWINE TYPE.

The practical data as given in Trautwine, published by Wiley & Sons, is used almost universally by engineers. (See Fig. 354.)

"When the backing is deposited loosely, as usual, as when dumped from carts, cars, etc.

"Wall of cut stone or of first-class large-ranged rubble in mortar, *ab*, .35 of its entire vertical height, *db*.

"Wall of good common scabbled mortar-rubble or brick .4 of its entire vertical height, *db*.

"Wall of well-scabbled dry rubble, .5 of its entire vertical height, *db*.

"With good masonry, however, we may take the height *ds* instead of *db*, and then the above proportions of *ds* will give a sufficient thickness at the ground-line, *os*.

"When the backing is somewhat consolidated in horizontal layers, each of these thicknesses may be reduced, but no rule can be given for this.

"The offset, *oe*, in front of the wall is not included in these thicknesses."

Probably the most important points to be considered in the design of any retaining wall are the foundation and outside toe pressures, the factor of safety against sliding, the method and character of the back fill, and proper drainage.

The pressure on the foundation should be arrived at from the same data that has been given for bridge piers, and the toe of the wall is put in as shown in Fig. 355, but to be not less than what would be safe should the material become softened up on account of improper drainage. Where the ground is soft, or liable to become so, piling should be used either driven vertically with the two rows under the toe spaced closer together (Fig. 411), or else with inclined brace piles under the toe as was used on a section of the seawall built at the Puget Sound Navy Yard.

In making the calculations of retaining walls the angle of repose as given in Table LI should be used, and the weight per cubic yard of various soils as given in Table LII. These same values may be used to determine the force tending to slide the wall at any section or upon the base. Where the wall is not properly sub-drained, not properly back filled, and not provided with weep holes, it is probable that water pressure may be exerted on the back of the wall, and in this case the calculations must be based on the hydrostatic pressure. The author recently constructed a retaining wall from plans which called for too narrow a base, and to make it safe two rows of weep holes were provided to drain off the water, and the back filling was done with broken rock so that the water would be properly drained off. The wall also had a very considerable surcharge from the covering of large concrete pipe carried on the ground at the top of the wall, and also had to sustain the pressure from the weight of the pipe filled with water. Where a wall carries a surcharge it must be carefully calculated, but roughly speaking the wall must have a 50 per cent. wider base than an ordinary retaining wall.

The specifications of the City of Seattle, for retaining walls, published by the Civil Engineering Department of the City, are very complete, and are given in full.

Foundation.—The foundation for any retaining wall is to be excavated to the depth called for on the plan, or to such depth as the City Engineer may determine is necessary to insure a proper footing. Where the location of the wall comes on soil which, in the opinion of the City Engineer, is not firm enough to insure its safety, piling or other suitable form of sub-foundation must be placed, as

the City Engineer will direct. The foundation pits shall at all times be kept dry and free from water by pumping or otherwise as may be directed. Where permanent drainage of the foundation, or other than that shown on the plan is necessary, a suitable tile or sewer-pipe drain is to be laid and connected with the sewer or suitable outlet.

Forms.—Forms for retaining walls to be constructed in accordance with the details given on the plan, or where no details are given, in a manner satisfactory to the City Engineer. They must be constructed of sound merchantable lumber thoroughly braced and stayed, so as to produce the finished surfaces true to line and grade, and free from wind or warp or objectionable depressions and projections. Lumber used to be evenly sized and free from knotholes or other imperfections affecting the finished work. Where monolithic construction is required, particular care must be taken to construct the forms of sufficient strength to prevent bulging.

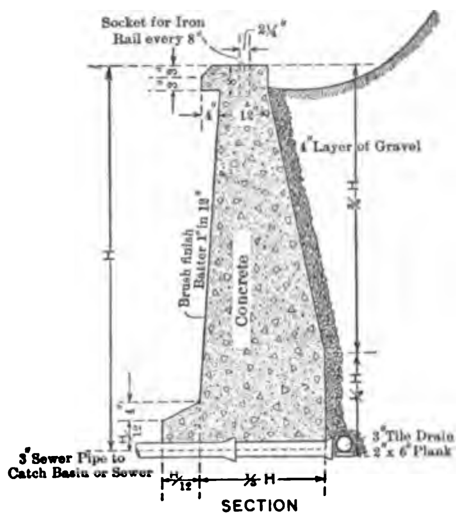


FIG. 355.—STANDARD RETAINING WALL, CITY OF SEATTLE.

All grooves, joints, mouldings, pilasters, panels and copings shall be formed true to line and dimension. Particular care to be exercised in constructing the forms for copings or other projecting parts of the wall or parapet that the same may be released and allowed to settle slightly after the concrete has partially set in order to prevent the expansion of the form from lifting or cracking the concrete at such projecting portions.

All forms to be so constructed that in stripping them from the finished work, the edges of moldings, etc., will not be defaced.

Concrete.—The concrete used in retaining walls is to be mixed in the proportions of one (1) part Portland cement, three (3) parts sand and six (6) parts gravel. The proportions of cement to the total aggregate used will be invariable, but the relative proportion of sand to gravel may be varied by the Engineer from time to time.

The concrete shall be deposited uniformly in layers but shall not be deposited in any part of the wall faster than it can be properly handled and spread into place. Depositing the material from a height into place, without properly remixing and spreading the same will not be permitted. Unless otherwise directed, the concrete must be mixed wet enough to readily spread and fill the forms but it shall not be mixed so wet that there is any tendency to wash the gravel free from the grout coating. All concrete must be thoroughly spaded as soon as deposited. The face of the wall is to be formed by spading back the gravel therefrom in such a manner as to leave a smooth cement finish. Before any concrete is deposited on top of a previous day's work, the latter shall be made rough by picking or chipping. All loose material and cement scum, or laitance, must be thoroughly removed, the surface washed clean and then grouted with neat cement. The scum, or laitance, to be removed before the concrete has set hard.

All walls shall be constructed as monoliths, where practical, that is, any section of a wall shall be deposited in one continuous operation, including the final finish at the top. Where monolithic construction is impractical, for the purpose of keeping each successive step of the work together, a recess six (6) inches deep and of a width equal to one-third the width of the wall shall be left at the end of each day's work for the entire length of such work in all walls where the cross-section is two (2) feet or more in thickness. In thinner walls, the contractor will be required to furnish and set steel dowel pins not less than three-quarters ($\frac{3}{4}$) of an inch square and two (2) feet long at intervals of not less than three (3) feet for the entire length of each day's work where the same is not brought to the finished height.

In all walls the forms, moldings, etc., along the fiinished sides must be kept cleaned of any dry mortar or concrete which may mar the finished appearance.

Joints.—Joints are to be made in all walls as indicated on the plan or as directed by the City Engineer. Where joints are required the wall shall be built in alternate sections. In the ends of each completed section a recess shall be provided, four (4) inches deep and of a width equal to one-third ($\frac{1}{3}$) the thickness of the wall, but not exceeding 1 foot, for the purpose of keying the sections of the wall together, or, steel dowel pins $\frac{3}{4}$ of an inch square and two (2) feet long can be set at intervals of two (2) feet, as may be directed.

Before the intermediate sections are built the ends of the alternate

sections must be coated with one coat of expansion-joint material and four (4) layers of No. 2 tarred roofing felt, each layer of roofing felt being coated with pitch or asphalt as laid.

At the finished face of the wall, the joint shall end in a V-shaped groove two (2) inches wide and one (1) inch deep unless otherwise shown on the plan.

Finish.—As soon as the forms are stripped, the surface of the wall shall be gone over with a chipping hammer and all projections brought down to an even surface. All wires must be snipped to the surface of the wall and all holes, projections or rough spots pointed up with a mortar composed of 1 part cement to 2 parts sand. Care shall be taken in removing the forms that edges, molding, etc., are not damaged. The entire surface shall be wetted and then given a brush finish with a coat of cement grout composed of 1 part plaster of paris and 3 parts cement mixed with water to a consistency of thick cream or with a thin coat or neat cement grout, as the City Engineer directs.

Waterproofing.—The back of the wall is to be coated with tar pitch, asphalt or other approved substance. Unless otherwise directed such waterproofing will consist of two coats of the substance selected. The waterproofing must be applied hot and only on a dry surface.

Gravel.—A layer of coarse gravel not less than 4 inches in thickness shall be placed at the back of the wall for its entire height and will be paid for per cubic yard in place.

Tile Drain.—A tile drain of the size called for on the plan is to be placed at the back of the wall at the bottom and connected to the sewer where shown in the plan.

Backfilling.—The backfilling behind retaining walls is not to be made until the walls have been allowed to set two weeks or longer. The filling to be made in layers not exceeding 1 foot in thickness and thoroughly rammed. Filling in with loose earth and puddling the same will not be permitted except by express permission of the City Engineer.

Measurements.—The quantities of materials to be paid for in concrete retaining walls shall be the actual quantities in the completed work, the volumes to be determined by the prismoidal formula.

Payment for plain concrete retaining walls will include all necessary excavating, concrete, dowel pins, joints, backfilling, finishing the surface, moldings, and the furnishing, placing and removing of all necessary forms.

Piling for sub-foundation work, gravel, waterproofing, tile drain and sewer pipe will be paid for at the rates bid for the same.

In case no bid is taken for reinforcing steel, 6 cents a pound will be paid for any used.

Payment.—Payment for reinforcing steel will be in full for furnishing, bending, fitting and placing the same in the work as called for on the plan. The measurement of steel will be for the length called for on the plan or as the City Engineer may direct to be placed in the completed work.

TABLE LI—ANGLE OF REPOSE OF SOILS.

(Engineer's Year Book).

Material.	Angle.	Ratio of Base of Slope to Height
Clay, dry	29°	1.8 to 1
Clay, damp, well drained	45°	1 to 1
Clay, wet	16°	3.5 to 1
Earth, dry	29°	1.8 to 1
Earth, moist	45 to 49°	1 to 1 to .87 to 1
Earth, very wet	17°	3.27 to 1
Earth, punned	66 to 74°	.45 to 1 to .28 to 1
Gravel, clean	48°	.9 to 1
Gravel, with sand	26°	2 to 1
Sand, fine dry	37 to 31°	1.3 to 1 to 1.6 to 1
Sand, wet	26°	2 to 1
Sand, very wet	32°	1.6 to 1
Shingle, loose	39°	1.2 to 1
Peat	14 to 45°	4 to 1 to 1 to 1

TABLE LII.—WEIGHT PER CUBIC YARD OF SOILS.

Material.	Lbs.	Material.	Lbs.
Dry peat	840	Gravel	3000
Wet peat	1680	Wet sand	3140
Top soil	2240	Gravelly clay	3360
Dry sand	2470	Rough Gravel	3800
Common earth	2700	Gray chalk	4000
Sandy Loam	2700	Sandstone	4150
Marl	2910	Shale	4350
Clay	3000	Limestone	4500

CHAPTER XXVI

CEMENT AND CONCRETE

The use of some cementing material in building construction has been practiced from the earliest times, but the intelligent employment of cement may be said to date back only as far as the time of the Roman Empire, and Vitruvius in his writings explains fully the methods of its use and the quality of the materials that should be used in making mortar and concrete, although his ideas as to the cause of the cementing action, while describing the results that took place, were very far from the truth, as he had no conception of the chemical processes involved.

The earlier kinds of cement were natural mixtures or deposits of rock which when put through the manufacturing process had the proper composition to form cement. The great trouble, however, with cement of this kind is the lack of uniformity in the composition of the rock itself, so that while one lot of cement might be first-class, another lot made from identically the same quarry might be far from good.

The first real Portland cement that was ever manufactured, so far as known, was under a patent taken out by John Aspdin, of Leeds, England, in 1824, while the first Portland cement manufactured in Germany was made near Stettin in 1852. The manufacture of Portland cement was begun in the United States about 1875, and at the present time there are extensive manufactories in practically every portion of the United States, producing cement in many cases better than the imported. Large amounts of natural cement are manufactured in the United States, and they are first-class for use in making mortar for masonry work, in making concrete for concrete filling of rock-faced piers, for concrete that is used in foundation pits, and in any location where the concrete is not exposed to abrasion or the weather. But where a first-class piece of work is desired and one that is exposed, nothing should be used in concrete but Portland cement.

Natural deposits of chalk and clay must be found so that they will yield a product containing approximately 62.2 per cent. of lime, 28.2 per cent. of silica, and 9.6 per cent. of alumina, or about one-third of the alumina can be replaced by ferric oxide, giving a cement with a composition of 61.7 per cent. lime, 27.4 per cent. of silica, 7.5 per cent. alumina, and 3.4 per cent. of ferric oxide. As a matter of fact, however, the European cements contain not only the above elements, but small quantities of magnesia, potash, soda, sulphuric acid, sand, carbonic acid, and water.

The process of manufacture of Portland cement does not concern the engineer as a feature of the construction of foundations, so that the details will not be gone into here, but reference may be made to "Portland Cement," by C. D. Jameson, M. Am. Soc. C.E., and to many later works.

When the process of manufacture has been properly carried out, up to the point of grinding, it is necessary that it should be ground with the greatest of care, as upon this depends very largely its value as a cementing substance, and the finer it is ground the more quick-setting it will be—the usual fineness being so that not more than 5 per cent. will remain upon a sieve of 2500 meshes per inch. In some factories after the cement is ground it is sifted to insure the output being of proper fineness, but as this is an extra expense and apt to cause carelessness among the employees, it is better to be sure that the machinery grinds it properly in the first instance. Nearly all the large manufactories now have storehouses of large capacity in which the cement is stored for some time, usually thirty days, before shipment, which very greatly improves the quality of the cement, by slaking out the free lime. One of the most prominent American brands was shipped out for some years without having been stored previous to packing, as the demand was so great as to make it necessary to ship immediately, but it was found that a considerable amount of free lime was present and caused the cement to be so quicksetting as to be rejected on important work, so that the manufacturers were compelled to build storehouses and pack it for shipment after it had been stored for some time.

Many engineers require cement to be emptied out from the barrels or bags into a storehouse at the site of the work and become thoroughly mixed and aged before use, but in this country it is usually impracticable to do this, owing to the speed with which work is pushed through. The cement used to be packed for shipment in barrels, the shipping weight of which is practically 400 pounds gross. These barrels are lined with waterproof paper to keep the

cement dry. It is now packed in paper or burlap sacks containing from one-fourth to one-third of a barrel in each sack. Portland cement is always packed in burlap sacks, weighing 94 pounds per sack. This method of packing is all right if the cement is to be used soon after packing, or the sacks kept under cover.

For many years the English cements had such a reputation that they were more used than any other; but German manufacturers, realizing that this prestige had caused the English to become somewhat careless, supplanted to a large extent the English cements by simply guaranteeing their product, thus forcing the manufacturers

FIG. 356.—CEMENT TESTING-MACHINE.

of other countries to turn out a product of greater superiority. That now being manufactured in the United States (1913) is the equal of any in the world.

At the present time the methods of testing cements are so uniform that it is only necessary for the makers to furnish a cement that will stand the tests, to have any brand of Portland cement accepted on important work. One of the large standard testing-machines is shown in Fig. 356.

Probably the most used specification is that recommended by the Committee on the Testing of Cement of the American Society

of Civil Engineers, this having been revised and adopted by the American Society for Testing Materials as given in Appendix V, and a very complete specification of this character is given in the appendix from the specifications for the Topeka bridge constructed by Edwin Thacher, M. Am. Soc. C.E. But the same engineer has more recently prepared General Specifications for Concrete Steel Bridges, revising the Specifications for Cement, and they are quoted in full as being the most valuable specifications of this character published:

"All foundations shall be shown on plans, and conform to the dimensions marked thereon.

"Foundations on rock shall be prepared by removing all sand, mud, or other soft material, and by excavating the bed-rock in such manner as may be described or shown on drawings.

"Foundations on hardpan, gravel, gravel and clay, cemented sand, or other material intended to carry the load without piles, shall be excavated to the depth shown on plans.

"Foundations on piles, when not otherwise described, shall be inclosed in a permanent coffer-dam or crib, and be excavated to the depth shown on plans, and the piles shall be driven after the excavations are made. The spaces between the piles shall be filled with concrete, and in case it is found necessary to lay the concrete under water, proper appliances must be used to insure its being deposited with as little injury as possible.

"The piles shall be oak, yellow pine, or other wood that will stand the blow of the hammer, straight, sound, and cut from live timber; trimmed close, cut off square at the butt, and have all bark taken off.

"The piles shall not be less than 12 inches nor more than 16 inches in diameter at the large end, nor less than 10 inches in diameter at the small end, for piles having a length of 30 feet and under; for greater lengths the diameter of small end may be reduced 1 inch for each 10 feet of additional length down to a minimum of 7 inches.

"The piles shall not be loaded with a weight greater than given by the following formula:

$$L = 2wh \div (S+1),$$

in which L = safe load in pounds, w = weight of hammer in pounds, h = fall of hammer in feet, S = last penetration in inches.

"The number and arrangement of the piles for each foundation shall be shown on plans, and they shall be sawed off at the elevations shown.

"The following values shall be used in calculations:

Modulus of elasticity of concrete	1,400,000 lbs.
Maximum compression per square inch on concrete	500 lbs.
Maximum shear per square inch on concrete	100 lbs.
Maximum tension per square inch on concrete	50 lbs.

The above to be exclusive of temperature stresses.

"The cement shall be a true Portland cement, made by calcining a proper mixture of calcareous and clayey earths; and if required the contractor shall furnish a certified statement of the chemical composition of the cement and the raw materials from which it is manufactured.

"The fineness of the cement shall be such that at least 99 per cent. will pass through a sieve of 50 meshes per lineal inch, at least 90 per cent. will pass through a sieve of 100 meshes per lineal inch, and at least 70 per cent. will pass through a sieve of 200 meshes per lineal inch.

"Samples for testing may be taken from each and every barrel delivered unless otherwise specified. Tensile tests will be made on specimens prepared and maintained until tested at a temperature of not less than 60° F. Each specimen will have an area of one square inch at the breaking section, and after being allowed to harden in moist air for 24 hours will be immersed and maintained under water until tested.

"The sand used in preparing the test specimens shall be clean, sharp, crushed quartz, retained on a sieve of 30 meshes per square inch, and passing through a sieve of 20 meshes per square inch.

"No more than 23 to 27 per cent. of water shall be used in preparing the test specimens of neat cement, and in the case of test specimens of one cement and three sand no more than 11 or 12 per cent. of water by weight shall be used.

"Specimens prepared from neat cement shall after seven days develop a tensile strength of not less than 450 pounds per square inch. Specimens prepared from a mixture of 1 part cement and 3 parts sand, parts by weight, shall after seven days develop a tensile strength of not less than 160 pounds per square inch, and not less than 220 pounds per square inch after twenty-eight days. Specimens prepared from a mixture of 1 part cement and 3 parts sand, parts by weight, and immersed after twenty-four hours in water maintained at 176° F., shall not swell nor crack, and shall after seven days develop a tensile strength of not less than 160 pounds per square inch.

"Cement mixed neat with about 27 per cent. of water to form a stiff paste, shall after thirty minutes be appreciably indented by the end of a wire $\frac{1}{12}$ inch in diameter loaded to weigh one-quarter pound. Cement made into thin plates on glass plates shall not crack, scale nor warp under the following treatment: Three parts will be made and allowed to harden in moist air at from 60 to 70° F.; one of these will be subjected to water-vapor at 176° F. for three hours, after which it shall be immersed in hot water for forty-eight hours, another shall be placed in water at from 60 to 70° F., and the third shall be left in moist air.

"All cement shall be kept housed and dry until wanted in the work.

"The concrete shall be composed of clean hard broken stone, or gravel with irregular surface, clean sharp sand, and cement, mixed in the proportions hereafter specified. Whenever the amount of work to be done is sufficient to justify it, approved mixing-machines shall be used. The ingredients shall be placed in the machine in a dry state, and in the volumes specified and be thoroughly mixed, after which clean water shall be added and the mixing continued until the wet mixture is thorough and the mass uniform. No more water shall be used than the concrete will bear without quaking in ramming. The mixing must be made as rapidly as possible and the batch deposited in the work without delay.

"If the mixing is done by hand, the cement and sand shall first be thoroughly mixed dry in the proportions specified. The stone previously drenched with water shall then be deposited on this mixture. Clean water shall be added and the mass be thoroughly mixed and tunred over until each stone is covered with mortar and the batch shall be deposited without delay, and be thoroughly rammed until all voids are filled. The grades of concrete to be used are as follows: For the arches between skew-backs—1 part Portland cement, 2 parts sand, and 4 parts broken stone, or gravel, that will pass through a 1¼-inch ring.

"For the foundations, abutments, piers, and spandrels—1 part Portland cement, 4 parts sand, and 8 parts broken stone, or gravel, that will pass through a 2-inch ring.

"If concrete facing is used, it shall be composed of 1 part Portland cement and 2½ parts sand, and shall have a thickness of at least 1 inch on all arch soffits, arch faces, abutments, piers, spandrels, or other exposed surfaces.

"There must be no definite plane or surface of demarcation between the facing and the concrete backing. The facing and

backing must be deposited in the same layer, and be well rammed in place at the time same. If the arch faces, quoins, or other exposed surfaces are marked to represent masonry, such division marks shall be made by triangular strips 2 inches wide and 1 inch deep fastened to the casing in perfectly straight and parallel lines, and all projecting corners will be beveled to correspond.

"No plastering will be allowed on the exposed faces of the work, but the inside faces of the spandrel walls covered by the fill may be plastered with mortar having the same composition as specified for facing.

"All keystones, brackets, consoles, dentils, pedestals, hand-railing posts and panels, and other ornamental work when used, also curbs and gutters, shall be of the designs shown on plans, and be molded in suitable molds. The mortar for at least 1 inch thick shall consist of 1 part Portland cement and $2\frac{1}{2}$ parts sand, and when the size of the molding will admit, the interior may be composed of concrete of the same composition as specified for the arches. When pedestals, posts, or panels carry lamp-posts, a 4-inch wrought-iron pipe shall be built into the concrete from top to bottom, and at bottom shall be connected with a 3-inch pipe extending under the sidewalk and connected with gas-pipe or electric-wire conduit. The pipes shall have no sharp bends, all changes in direction being made by gentle curves.

"During warm and dry weather, all newly built concrete shall be well sprinkled with water for several days, or until it is well set.

"The volumes of cement, sand, and broken stone in all mixtures of mortar or concrete used in the work shall be measured loose.

"In connecting concrete already set with new concrete the surface shall be cleaned and roughened, and mopped with a mortar composed of 1 part Portland cement and 1 part sand, to cement the parts together.

"The concrete for the arches shall be started simultaneously from both ends of the arch, and be built in longitudinal sections wide enough to inclose at least two steel ribs, and of sufficient width to constitute a day's work. The concrete shall be deposited in layers, each layer being well rammed in place before the previously deposited layer has had time to partially set. The work shall proceed continuously day and night if necessary to complete each longitudinal section. These sections while being built shall be held in place by substantial timber forms, normal to the centering and parallel to

each other, and these forms shall be removed when the section has set sufficiently to admit of it. The sections shall be connected as specified and also by steel clamps or rib connections built into the concrete."

The average requirements of the United States Government Engineers are that 95½ per cent. shall pass through a 2500-mesh sieve and 84 per cent. through a 10,000-mesh sieve. This requirement is considerably lower than is called for by many of the cities in the United States, the average of nine cities taken for this purpose requiring 97 per cent. to pass through a 2500-mesh sieve and 89 per cent. through a 10,000-mesh sieve. The United States Government Engineers require the cement to test up to 402 pounds per square inch for seven-day tests of neat cement, and up to 119 pounds for a seven-day test of 3 to 1 briquettes; while the same nine cities mentioned above show an average requirement of 388 pounds per square inch for seven-day tests of neat cement, and 134 pounds for seven-day tests of 3 to 1 briquettes.

The United States Government Navy Specifications for Portland Cement are very thorough and are given in Appendix VIII.

The proportion of concrete and the methods of depositing it have been fully covered in previous chapters, but the practical use of cement makes it desirable to have tables of concrete mixtures of various proportions, which give the amount of cement in barrels, the amount of sand in yards, and the amount of broken stone or gravel in yards, to use to make one yard of tamped concrete.

One of the first tables of this sort to be published, which had been checked by practice, was in Vol. 42 of the Transactions of the American Society of Civil Engineers, in a discussion by the author, of a paper on The Theory of Concrete. (Table LIX.) More complete tables of this kind have been worked out from theory, experiment, and practice by Edwin Thacher, M. Am. Soc. C.E., and with his permission are reprinted here. (Tables LIII and LIV.)

The discussion by the author on the amount of material in different concretes is given in full, as it forms an interesting comparison with the tables prepared by Thacher. (Table LIX.) The column with stone having 0.4 voids agrees very closely with Thacher's table for concrete with 2½-inch stone, the agreement being so close as to make the tables practically identical.

" This subject has impressed the speaker as being a very important one, for the reason that in figuring on work he has, as a rule, found that engineers and contractors know only approximately the quan-

TABLE LIII.—MATERIAL FOR CONCRETE.

Concrete with "Hazelnut" Stone.						Concrete with Stone 2½" and Under.					
Proportions of Mixture.			Required for 1 Cubic Yard.			Proportions of Mixture.			Required for 1 Cubic Yard.		
Cement.	Sand.	Stone.	Cement, Barrels.	Sand. Cubic Yards.	Stone. Cubic Yards.	Cement.	Sand.	Stone.	Cement. Barrels.	Sand, Cubic Yards.	Stone, Cubic Yards.
1	1	2.0	2.57	0.39	0.78	1	1	2.0	2.63	0.40	0.80
1	1	2.5	2.29	0.35	0.70	1	1	2.5	2.34	0.36	0.89
1	1	3.0	2.06	0.31	0.94	1	1	3.0	2.10	0.32	0.96
1	1	3.5	1.84	0.28	0.98	1	1	3.5	1.88	0.29	1.00
1	1.5	2.5	2.05	0.47	0.78	1	1.5	2.5	2.09	0.48	0.80
1	1.5	3.0	1.85	0.42	0.84	1	1.5	3.0	1.90	0.43	0.87
1	1.5	3.5	1.72	0.39	0.91	1	1.5	3.5	1.74	0.40	0.93
1	1.5	4.0	1.57	0.36	0.96	1	1.5	4.0	1.61	0.37	0.98
1	1.5	4.5	1.43	0.33	0.98	1	1.5	4.5	1.46	0.33	1.00
1	2.0	3.0	1.70	0.52	0.77	1	2.0	3.0	1.73	0.53	0.79
1	2.0	3.5	1.57	0.48	0.83	1	2.0	2.5	1.61	0.49	0.85
1	2.0	4.0	1.46	0.44	0.89	1	2.0	4.0	1.48	0.45	0.90
1	2.0	4.5	1.36	1.42	0.93	1	2.0	4.5	1.38	0.42	0.95
1	2.0	5.0	1.27	0.39	0.97	1	2.0	5.0	1.29	0.39	0.98
1	2.5	3.5	1.45	0.55	0.77	1	2.5	3.5	1.48	0.56	0.79
1	2.5	4.0	1.35	0.52	0.82	1	2.5	4.0	1.38	0.53	0.84
1	2.5	4.5	1.27	0.48	0.87	1	2.5	4.5	1.29	0.49	0.88
1	2.5	5.0	1.19	0.46	0.91	1	2.5	5.0	1.21	0.46	0.92
1	2.5	5.5	1.13	0.43	0.94	1	2.5	5.5	1.15	0.44	0.96
1	2.5	6.0	1.07	0.41	0.97	1	2.5	6.0	1.07	0.41	0.98
1	3.0	4.0	1.26	0.58	0.77	1	3.0	4.0	1.28	0.58	0.78
1	3.0	4.5	1.18	0.54	0.81	1	3.0	4.5	1.20	0.55	0.82
1	3.0	5.0	1.11	0.51	0.85	1	3.0	5.0	1.14	0.52	0.87
1	3.0	5.5	1.06	0.48	0.89	1	3.0	5.5	1.07	1.49	0.90
1	3.0	6.0	1.01	0.46	0.92	1	3.0	6.0	1.02	0.47	0.93
1	3.0	6.5	0.96	0.44	0.95	1	3.0	6.5	0.98	0.44	0.96
1	3.0	7.0	0.91	0.42	0.97	1	3.0	7.0	0.92	0.42	0.98
1	3.5	5.0	1.05	0.56	0.80	1	3.5	5.0	1.07	0.57	0.82
1	3.5	5.5	1.00	0.53	0.84	1	3.5	5.5	1.02	0.54	0.85
1	3.5	6.0	0.95	0.50	0.87	1	3.5	6.0	0.97	0.51	0.89
1	3.5	6.5	0.92	0.49	0.91	1	3.5	6.5	0.93	0.49	0.92
1	3.5	7.0	0.87	0.47	0.93	1	3.5	7.0	0.89	0.47	0.95
1	3.5	7.5	0.84	0.45	0.96	1	3.5	7.5	0.85	0.45	0.98
1	3.5	8.0	0.80	0.42	0.97	....	....	....	....	....	....
1	4.0	6.0	0.90	0.55	0.82	1	4.0	6.0	0.92	0.56	0.84
1	4.0	6.5	0.87	0.53	0.85	1	4.0	6.5	0.88	0.53	0.87
1	4.0	7.0	0.83	0.51	0.89	1	4.0	7.0	0.84	0.51	0.90
1	4.0	7.5	0.80	0.49	0.91	1	4.0	7.5	0.81	0.50	0.93
1	4.0	8.0	0.77	0.47	0.93	1	4.0	8.0	0.78	0.48	0.95
1	4.0	8.5	0.74	0.45	0.95	1	4.0	8.5	0.76	0.46	0.98
1	4.0	9.0	0.71	0.43	0.97	....	....	....	....	....	....
1	5.0	9.0	0.66	0.50	0.90	1	5.0	9.0	0.67	0.52	0.93
1	5.0	10.0	0.62	0.47	0.95	1	5.0	10.0	0.63	0.48	0.96
1	6.0	11.0	0.55	0.51	0.93	1	6.0	11.0	0.56	0.52	0.94
1	6.0	12.0	0.52	0.48	0.95	1	6.0	12.0	0.54	0.49	0.98
1	7.0	13.0	0.47	0.50	0.93	1	7.0	13.0	0.48	0.51	0.95
1	7.0	14.0	0.45	0.48	0.96	1	7.0	14.0	0.46	0.49	0.98

Compiled by Edwin Thacher, M. Am. Soc. C.E.

TABLE LIV.—MATERIAL FOR CONCRETE (CONTINUED).

Concrete with 2½" Stone.						Concrete with Gravel ¾" and Under.					
Proportions of Mixture.			Required for 1 Cubic Yard.			Proportions of Mixture.			Required for 1 Cubic Yard.		
Cement.	Sand.	Stone.	Cement. Barrels.	Sand. Cubic Yards.	Stone. Cubic Yards.	Cement.	Sand.	Gravel.	Cement. Barrels.	Sand. Cubic Yards.	Gravel. Cubic Yards.
1	1	2.0	2.72	0.41	0.83	1	1	2.5	2.10	0.32	0.80
1	1	2.5	2.41	0.37	0.92	1	1	3.0	1.89	0.29	0.86
1	1	3.0	2.16	0.33	0.98	1	1	3.5	1.71	0.26	0.91
....	....	....	....	....	....	1	1	4.0	1.55	0.24	0.94
1	1.5	2.5	2.16	0.49	0.82	1	1.5	3.0	1.71	0.39	0.78
1	1.5	3.0	1.96	0.45	0.89	1	1.5	3.5	1.57	0.36	0.83
1	1.5	3.5	1.79	0.41	0.96	1	1.5	4.0	1.46	0.33	0.88
1	1.5	4.0	1.64	0.38	1.00	1	1.5	4.5	1.34	0.31	0.91
....	....	....	....	....	....	1	1.5	5.0	1.24	0.28	0.94
1	2.0	3.0	1.78	0.54	0.81	1	2.0	3.5	1.44	0.44	0.77
1	2.0	3.5	1.66	0.50	0.88	1	2.0	4.0	1.34	0.41	0.81
1	2.0	4.0	1.53	0.47	0.93	1	2.0	4.5	1.26	0.38	0.86
1	2.0	4.5	1.43	0.43	0.98	1	2.0	5.0	1.17	0.36	0.89
....	....	....	....	....	....	1	2.0	6.0	1.03	0.31	0.94
1	2.5	3.5	1.51	0.58	0.81	1	2.5	4.0	1.24	0.47	0.75
1	2.5	4.0	1.42	0.54	0.87	1	2.5	4.5	1.16	0.44	0.80
1	2.5	4.5	1.33	0.51	0.91	1	2.5	5.0	1.10	0.42	0.83
1	2.5	5.0	1.26	0.48	0.96	1	2.5	5.5	1.03	0.39	0.86
1	2.5	5.5	1.18	0.44	0.99	1	2.5	6.0	0.98	0.37	0.89
....	....	....	....	....	....	1	2.5	7.0	0.88	0.33	0.93
1	3.0	4.0	1.32	0.60	0.80	1	3.0	5.0	1.03	0.47	0.78
1	3.0	4.5	1.24	0.57	0.85	1	3.0	5.5	0.97	0.44	0.81
1	3.0	5.0	1.17	0.54	0.89	1	3.0	6.0	0.92	0.42	0.84
1	3.0	5.5	1.11	0.51	0.93	1	3.0	6.5	0.88	0.40	0.87
1	3.0	6.0	1.06	0.48	0.97	1	3.0	7.0	0.84	0.38	0.89
....	....	....	....	....	....	1	3.0	7.5	0.80	0.37	0.91
....	....	....	....	....	....	1	3.0	8.0	0.76	0.35	0.93
1	3.5	5.0	1.11	0.59	0.85	1	3.5	6.0	0.88	0.46	0.80
1	3.5	5.5	1.06	0.56	0.89	1	3.5	6.5	0.83	0.44	0.82
1	3.5	6.0	1.00	0.53	0.92	1	3.5	7.0	0.80	0.43	0.85
1	3.5	6.5	0.96	0.51	0.95	1	3.5	7.5	0.76	0.41	0.87
1	3.5	7.0	0.91	0.49	0.98	1	3.5	8.0	0.73	0.39	0.89
....	....	....	....	....	....	1	3.5	8.5	0.71	0.38	0.91
....	....	....	....	....	....	1	3.5	9.0	0.68	0.36	0.92
1	4.0	6.0	0.95	0.58	0.87	1	4.0	7.0	0.77	0.47	0.81
1	4.0	6.5	0.91	0.55	0.90	1	4.0	7.5	0.73	0.44	0.83
1	4.0	7.0	0.87	0.53	0.93	1	4.0	8.0	0.71	0.43	0.86
1	4.0	7.5	0.84	0.51	0.96	1	4.0	8.5	0.68	0.42	0.88
1	4.0	8.0	0.81	0.49	0.98	1	4.0	9.0	0.65	0.40	0.89
....	....	....	....	....	....	1	4.0	9.5	0.63	0.38	0.91
....	....	....	....	....	....	1	4.0	10.0	0.61	0.37	0.93
1	5.5	8.0	0.74	0.57	0.91	1	5.0	10.0	0.57	0.43	0.87
1	5.0	9.0	0.70	0.53	0.96	1	5.0	12.0	0.51	0.38	0.92
1	6.0	9.0	0.65	0.59	0.89	1	6.0	12.0	0.48	0.44	0.88
1	6.0	10.0	0.62	0.56	0.93	1	6.0	14.0	0.43	0.40	0.92
1	7.0	11.0	0.54	0.51	0.91	1	7.0	14.0	0.42	0.44	0.88
1	7.0	12.0	0.52	0.55	0.95	1	7.0	16.0	0.38	0.40	0.92

Compiled by Edwin Thacher, M. Am. Soc. C.E.

TABLE LV.—QUANTITIES OF MATERIALS FOR ONE CUBIC YARD OF RAMMED CONCRETE.[1]

BASED ON A BARREL OF 3.8 CUBIC FEET.

Proportions by parts: Cement	Sand	Stone	Proportions by volume: Packed Cement bbl	Loose Sand cu. ft.	Loose Stone cu. ft.	Volume of mortar in terms of percentage of volume of stone %	50%* Cement bbl	50%* Sand cu. yd.	50%* Stone cu. yd.	45%† Cem'nt bbl	45%† Sand cu. yd.	45%† Stone cu. yd.	40%‡ Cement bbl	40%‡ Sand cu. yd.	40%‡ Stone cu. yd.	30%§ Cement bbl	30%§ Sand cu. yd.	30%§ Stone cu. yd.	20%§ Cement bbl	20%§ Sand cu. yd.	20%§ Stone cu. yd.
1		1	1		3.8	94	5.09		0.72	4.90		0.69	4.73		0.67	4.33		0.61	4.02		0.57
1		2	1		7.6	51	3.67		1.03	3.48		0.98	3.30		0.93	2.93		0.82	2.65		0.75
1		3	1		11.4	36				2.69		1.14	2.54		1.07	2.22		0.94	1.98		0.84
1		4	1		15.2	29										1.78		1.00	1.58		0.89
1		5	1		19.0	25										1.40		1.05	1.31		0.92
1		6	1		22.8	22										1.28		1.08	1.12		0.95
1		7	1		26.6	20													0.98		0.97
1		8	1		30.4	19													0.87		0.98
1		9	1		34.2	18													0.78		0.99
1		10	1		38.0	17													0.71		1.00
1		11	1		41.8	16													0.65		1.01
1		12	1		45.5	15													0.60		1.01
1	1	1½	1	3.8	5.7	99	3.19	0.45	0.67	3.08	0.43	0.65	2.97	0.42	0.63	2.78	0.39	0.50	2.62	0.37	0.55
1	1	2	1	3.8	7.6	75	2.85	0.40	0.80	2.73	0.38	0.77	2.62	0.37	0.74	2.43	0.34	0.68	2.26	0.32	0.64
1	1	2½	1	3.8	9.5	61	2.57	0.36	0.90	2.45	0.34	0.86	2.34	0.33	0.82	2.15	0.30	0.76	1.99	0.28	0.70
1	1	3	1	3.8	11.4	51	2.34	0.33	0.99	2.22	0.31	0.94	2.12	0.30	0.90	1.93	0.27	0.82	1.77	0.25	0.75
1	1½	2	1	5.7	7.6	93	2.49	0.53	0.70	2.40	0.51	0.68	2.31	0.49	0.65	2.16	0.46	0.61	2.03	0.43	0.57
1	1½	2½	1	5.7	9.5	76	2.27	0.48	0.80	2.18	0.46	0.77	2.00	0.44	0.74	1.94	0.41	0.68	1.80	0.38	0.63
1	1½	3	1	5.7	11.4	64	2.09	0.44	0.88	2.00	0.42	0.84	1.01	0.40	0.81	1.76	0.37	0.74	1.63	0.34	0.69
1	1½	3½	1	5.7	13.3	55	1.94	0.41	0.96	1.84	0.39	0.91	1.76	0.37	0.87	1.61	0.34	0.79	1.48	0.31	0.73
1	1½	4	1	5.7	15.2	49	1.80	0.38	1.01	1.71	0.36	0.96	1.63	0.34	0.92	1.48	0.31	0.83	1.36	0.29	0.77
1	1½	4½	1	5.7	17.1	44	1.69	0.36	1.07	1.60	0.34	1.01	1.51	0.32	0.96	1.37	0.29	0.87	1.25	0.26	0.79
1	1½	5	1	5.7	19.0	40	1.59	0.34	1.12	1.50	0.32	1.06	1.42	0.30	1.00	1.28	0.27	0.90	1.17	0.25	0.82
1	2	3	1	7.6	11.4	75	1.89	0.53	0.80	1.81	0.51	0.76	1.74	0.49	0.74	1.61	0.45	0.68	1.50	0.42	0.63
1	2	3½	1	7.6	13.3	65	1.76	0.49	0.87	1.68	0.47	0.83	1.61	0.45	0.79	1.48	0.42	0.73	1.38	0.39	0.68
1	2	4	1	7.6	15.2	57	1.65	0.46	0.93	1.57	0.44	0.88	1.50	0.42	0.84	1.38	0.39	0.78	1.27	0.36	0.72
1	2	4½	1	7.6	17.1	51	1.55	0.44	0.98	1.48	0.42	0.94	1.41	0.40	0.89	1.28	0.36	0.81	1.18	0.33	0.75
1	2	5	1	7.6	19.0	47	1.47	0.41	1.03	1.39	0.39	0.98	1.32	0.37	0.93	1.20	0.34	0.84	1.10	0.31	0.77
1	2	5½	1	7.6	20.9	43	1.39	0.39	1.08	1.31	0.37	1.01	1.25	0.35	0.97	1.13	0.32	0.87	1.03	0.29	0.80
1	2	6	1	7.6	22.8	40	1.32	0.37	1.11	1.25	0.35	1.06	1.18	0.33	1.00	1.06	0.30	0.89	0.97	0.27	0.82
1	2½	3	1	9.5	11.4	87	1.72	0.61	0.73	1.66	0.58	0.70	1.60	0.56	0.68	1.49	0.52	0.63	1.40	0.49	0.59
1	2½	3½	1	9.5	13.3	75	1.62	0.57	0.80	1.55	0.55	0.76	1.49	0.52	0.73	1.38	0.49	0.68	1.29	0.45	0.64
1	2½	4	1	9.5	15.2	66	1.52	0.54	0.86	1.46	0.51	0.82	1.40	0.49	0.79	1.29	0.45	0.73	1.19	0.42	0.67
1	2½	4½	1	9.5	17.1	60	1.44	0.51	0.91	1.37	0.48	0.87	1.31	0.46	0.83	1.20	0.42	0.76	1.11	0.39	0.70
1	2½	5	1	9.5	19.0	54	1.37	0.48	0.96	1.30	0.46	0.92	1.24	0.44	0.87	1.13	0.40	0.80	1.04	0.37	0.73
1	2½	5½	1	9.5	20.9	49	1.30	0.46	1.01	1.23	0.43	0.95	1.17	0.41	0.91	1.07	0.38	0.83	0.98	0.34	0.76
1	2½	6	1	9.5	22.8	46	1.24	0.44	1.05	1.17	0.41	0.99	1.11	0.39	0.94	1.01	0.36	0.85	0.92	0.32	0.78
1	2½	6½	1	9.5	24.7	42	1.18	0.42	1.08	1.12	0.39	1.02	1.06	0.37	0.97	0.96	0.34	0.88	0.88	0.31	0.80
1	2½	7	1	9.5	26.6	40	1.13	0.40	1.11	1.07	0.38	1.05	1.01	0.36	0.99	0.91	0.32	0.90	0.83	0.29	0.82
1	3	4	1	11.4	15.2	76	1.42	0.60	0.80	1.36	0.57	0.77	1.30	0.55	0.73	1.21	0.51	0.68	1.12	0.47	0.63
1	3	4½	1	11.4	17.1	68	1.34	0.57	0.85	1.28	0.54	0.81	1.23	0.52	0.78	1.13	0.48	0.72	1.05	0.44	0.66
1	3	5	1	11.4	19.0	61	1.28	0.54	0.90	1.22	0.52	0.86	1.17	0.49	0.82	1.07	0.45	0.75	0.99	0.42	0.70
1	3	5½	1	11.4	20.9	56	1.22	0.52	0.94	1.16	0.49	0.90	1.11	0.47	0.86	1.01	0.43	0.78	0.93	0.39	0.72
1	3	6	1	11.4	22.8	52	1.16	0.40	0.98	1.11	0.47	0.94	1.05	0.44	0.89	0.96	0.41	0.81	0.88	0.37	0.74
1	3	6½	1	11.4	24.7	48	1.12	0.47	1.02	1.06	0.45	0.97	1.01	0.43	0.92	0.92	0.39	0.84	0.84	0.35	0.77
1	3	7	1	11.4	26.6	45	1.07	0.45	1.05	1.01	0.43	0.99	0.96	0.40	0.95	0.87	0.37	0.86	0.80	0.34	0.79
1	3	7½	1	11.4	28.5	42	1.03	0.44	1.09	0.97	0.41	1.02	0.92	0.39	0.97	0.83	0.35	0.88	0.76	0.32	0.80
1	3	8	1	11.4	30.4	40	0.99	0.42	1.11	0.93	0.39	1.05	0.88	0.37	0.99	0.80	0.34	0.90	0.73	0.31	0.82
1	4	5	1	15.2	19.0	76	1.13	0.64	0.80	1.08	0.61	0.76	1.04	0.59	0.73	0.96	0.54	0.68	0.90	0.51	0.63
1	4	6	1	15.2	22.8	64	1.04	0.59	0.88	0.99	0.56	0.84	0.95	0.54	0.80	0.87	0.49	0.73	0.81	0.46	0.68
1	4	7	1	15.2	26.6	55	0.96	0.54	0.95	0.92	0.52	0.91	0.88	0.50	0.87	0.80	0.45	0.79	0.74	0.42	0.73
1	4	8	1	15.2	30.4	49	0.90	0.51	1.01	0.85	0.48	0.96	0.81	0.46	0.91	0.74	0.42	0.83	0.68	0.38	0.77
1	4	9	1	15.2	34.2	44	0.84	0.47	1.06	0.80	0.45	1.01	0.76	0.43	0.96	0.68	0.38	0.86	0.63	0.35	0.80
1	4	10	1	15.2	38.0	40	0.79	0.44	1.11	0.75	0.42	1.06	0.71	0.40	1.00	0.64	0.36	0.90	0.58	0.33	0.82
1	5	10	1	19.0	38.0	47	0.73	0.52	1.03	0.69	0.49	0.97	0.66	0.46	0.93	0.60	0.42	0.84	0.55	0.39	0.77
1	6	12	1	22.8	45.5	46	0.62	0.52	1.04	0.58	0.49	0.98	0.56	0.47	0.94	0.50	0.42	0.84	0.46	0.39	0.78

Note. — Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†*Use 45% columns for average conditions* and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

[1] From "Concrete, Plain and Reinforced," Taylor and Thompson. Copyright, 1905 and 1909, by Frederick W. Taylor. John Wiley & Sons, Inc., New York.

TABLE LVI.—QUANTITIES OF MATERIAL FOR ONE CUBIC YARD OF RAMMED CONCRETE.[1]

BASED ON A BARREL OF 4 CUBIC FEET.

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUMES			Volume of mortar in terms of percentage of volume of stone	PERCENTAGES OF VOIDS IN BROKEN STONE OR GRAVEL: 50%*			45%†			40%‡			30%§			20%§		
Cement	Sand	Stone	Packed Cement bbl	Loose Sand cu. ft.	Loose Stone cu. ft.	%	Cement bbl	Sand cu. yd.	Stone cu. yd	Cement bbls	Sand cu. yd.	Stone cu. yd.	Cement bbl	Sand cu. yd.	Stone cu. yd.	Cement bbl	Sand cu yd.	Stone cu. yd.	Cement bbl	Sand cu. yd.	Stone cu. yd.
1		1	1		4	89	4.99		0.74	4.80		0.71	4.62		0.69	4.23		0.63	3.91		0.58
1		2	1		8	49	3.57		1.06	3.37		1.00	3.20		0.95	2.84		0.84	2.56		0.76
1		3	1		12	35				2.60		1.16	2.45		1.09	2.13		0.95	1.90		0.84
1		4	1		16	28										1.71		1.01	1.51		0.89
1		5	1		20	24										1.43		1.06	1.26		0.93
1		6	1		24	22										1.22		1.08	1.07		0.95
1		7	1		28	20													0.94		0.98
1		8	1		32	18													0.83		0.98
1		9	1		36	17													0.75		1.00
1		10	1		40	16													0.68		1.01
1		11	1		44	15													0.62		1.01
1		12	1		48	15													0.57		1.01
1	1	1½	1	4	6	96	3.08	0.46	0.68	2.97	0.44	0.66	2.87	0.42	0.64	2.69	0.40	0.60	2.53	0.38	0.56
1	1	2	1	4	8	73	2.74	0.41	0.81	2.63	0.39	0.78	2.52	0.37	0.75	2.33	0.34	0.69	2.17	0.32	0.64
1	1	2½	1	4	10	59	2.47	0.37	0.91	2.35	0.35	0.87	2.25	0.33	0.83	2.06	0.31	0.76	1.90	0.28	0.71
1	1	3	1	4	12	50	2.25	0.33	1.00	2.13	0.32	0.95	2.03	0.30	0.90	1.85	0.27	0.82	1.70	0.25	0.76
1	1½	2	1	6	8	92	2.39	0.53	0.71	2.30	0.51	0.68	2.22	0.49	0.66	2.07	0.46	0.61	1.94	0.43	0.58
1	1½	2½	1	6	10	74	2.18	0.48	0.81	2.09	0.46	0.77	2.01	0.45	0.74	1.86	0.41	0.69	1.73	0.38	0.64
1	1½	3	1	6	12	62	2.01	0.45	0.89	1.91	0.42	0.85	1.83	0.41	0.81	1.68	0.37	0.75	1.56	0.35	0.69
1	1½	3½	1	6	14	54	1.86	0.41	0.96	1.77	0.39	0.92	1.68	0.37	0.87	1.54	0.34	0.80	1.42	0.32	0.74
1	1½	4	1	6	16	48	1.73	0.38	1.03	1.64	0.36	0.97	1.56	0.35	0.92	1.42	0.32	0.84	1.30	0.29	0.77
1	1½	4½	1	6	18	43	1.62	0.36	1.08	1.53	0.34	1.02	1.45	0.32	0.97	1.31	0.29	0.87	1.20	0.27	0.80
1	1½	5	1	6	20	39	1.52	0.34	1.13	1.43	0.32	1.06	1.35	0.30	1.00	1.22	0.27	0.90	1.11	0.25	0.82
1	2	3	1	8	12	74	1.81	0.54	0.80	1.74	0.52	0.77	1.67	0.50	0.74	1.54	0.46	0.68	1.44	0.43	0.64
1	2	3½	1	8	14	64	1.69	0.50	0.88	1.61	0.48	0.83	1.54	0.46	0.80	1.42	0.42	0.74	1.31	0.39	0.68
1	2	4	1	8	16	56	1.58	0.47	0.94	1.51	0.45	0.89	1.44	0.43	0.85	1.32	0.39	0.78	1.21	0.36	0.72
1	2	4½	1	8	18	51	1.49	0.44	0.99	1.41	0.42	0.94	1.34	0.40	0.89	1.23	0.36	0.82	1.13	0.34	0.75
1	2	5	1	8	20	46	1.40	0.42	1.04	1.33	0.39	0.98	1.26	0.37	0.93	1.15	0.34	0.85	1.05	0.31	0.78
1	2	5½	1	8	22	42	1.33	0.39	1.08	1.26	0.37	1.03	1.19	0.35	0.97	1.08	0.32	0.88	0.98	0.29	0.80
1	2	6	1	8	24	39	1.26	0.37	1.12	1.19	0.35	1.06	1.13	0.34	1.00	1.02	0.30	0.91	0.93	0.28	0.83
1	2½	3	1	10	12	86	1.65	0.61	0.73	1.59	0.59	0.71	1.53	0.57	0.68	1.42	0.52	0.63	1.33	0.49	0.59
1	2½	3½	1	10	14	75	1.55	0.57	0.80	1.48	0.55	0.77	1.42	0.52	0.74	1.32	0.49	0.68	1.23	0.46	0.64
1	2½	4	1	10	16	66	1.46	0.54	0.87	1.39	0.51	0.82	1.33	0.49	0.79	1.23	0.46	0.73	1.14	0.42	0.68
1	2½	4½	1	10	18	59	1.38	0.51	0.92	1.31	0.48	0.87	1.25	0.46	0.83	1.15	0.43	0.77	1.06	0.39	0.71
1	2½	5	1	10	20	54	1.31	0.48	0.97	1.24	0.46	0.92	1.18	0.44	0.87	1.08	0.40	0.80	0.99	0.37	0.73
1	2½	5½	1	10	22	49	1.24	0.46	1.01	1.18	0.44	0.96	1.12	0.41	0.91	1.02	0.38	0.83	0.93	0.34	0.76
1	2½	6	1	10	24	45	1.18	0.44	1.05	1.12	0.41	1.00	1.06	0.39	0.94	0.96	0.36	0.85	0.88	0.33	0.78
1	2½	6½	1	10	26	42	1.13	0.42	1.09	1.07	0.40	1.03	1.01	0.37	0.97	0.92	0.34	0.89	0.84	0.31	0.81
1	2½	7	1	10	28	39	1.08	0.40	1.12	1.02	0.38	1.06	0.96	0.36	1.00	0.87	0.32	0.90	0.79	0.29	0.82
1	3	4	1	12	16	75	1.35	0.60	0.80	1.30	0.58	0.77	1.25	0.56	0.74	1.15	0.51	0.68	1.08	0.48	0.64
1	3	4½	1	12	18	67	1.28	0.57	0.85	1.23	0.55	0.82	1.18	0.52	0.79	1.08	0.48	0.72	1.01	0.45	0.67
1	3	5	1	12	20	60	1.22	0.54	0.90	1.16	0.52	0.86	1.11	0.49	0.82	1.02	0.45	0.76	0.94	0.42	0.70
1	3	5½	1	12	22	55	1.16	0.52	0.95	1.11	0.49	0.90	1.06	0.47	0.86	0.97	0.43	0.79	0.89	0.40	0.72
1	3	6	1	12	24	50	1.11	0.49	0.99	1.06	0.47	0.94	1.01	0.45	0.90	0.92	0.41	0.82	0.84	0.37	0.75
1	3	6½	1	12	26	48	1.06	0.47	1.02	1.01	0.45	0.97	0.96	0.43	0.92	0.87	0.39	0.84	0.80	0.36	0.77
1	3	7	1	12	28	44	1.02	0.45	1.06	0.97	0.43	1.01	0.92	0.41	0.95	0.83	0.37	0.86	0.76	0.34	0.79
1	3	7½	1	12	30	42	0.98	0.44	1.09	0.93	0.41	1.03	0.88	0.39	0.98	0.79	0.35	0.88	0.73	0.32	0.81
1	3	8	1	12	32	39	0.94	0.42	1.11	0.89	0.40	1.05	0.84	0.37	1.00	0.76	0.34	0.90	0.69	0.31	0.82
1	4	5	1	16	20	75	1.08	0.64	0.80	1.03	0.61	0.76	0.99	0.59	0.73	0.92	0.55	0.68	0.86	0.51	0.64
1	4	6	1	16	24	63	0.99	0.59	0.88	0.95	0.56	0.84	0.91	0.54	0.81	0.83	0.49	0.74	0.77	0.46	0.68
1	4	7	1	16	28	55	0.92	0.54	0.95	0.88	0.52	0.91	0.83	0.49	0.86	0.76	0.45	0.79	0.70	0.42	0.73
1	4	8	1	16	32	48	0.86	0.51	1.02	0.81	0.48	0.96	0.77	0.46	0.91	0.70	0.42	0.83	0.64	0.38	0.76
1	4	9	1	16	36	43	0.80	0.47	1.07	0.76	0.45	1.01	0.72	0.43	0.96	0.65	0.39	0.87	0.60	0.36	0.80
1	4	10	1	16	40	40	0.75	0.44	1.11	0.71	0.42	1.05	0.67	0.40	0.99	0.61	0.36	0.90	0.55	0.33	0.81
1	5	10	1	20	40	47	0.70	0.52	1.04	0.66	0.49	0.98	0.63	0.47	0.93	0.57	0.42	0.84	0.52	0.38	0.77
1	6	12	1	24	48	46	0.59	0.52	1.05	0.56	0.50	1.00	0.53	0.47	0.94	0.48	0.43	0.85	0.44	0.30	0.78

NOTE.— Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†Use 45% columns for average conditions and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures

[1] From "Concrete, Plain and Reinforced," Taylor and Thompson. Copyright, 1905 and 1909, by Frederick W. Taylor. John Wiley & Sons, Inc., New York.

TABLE LVII.—VOLUME OF CONCRETE BASED ON A BARREL OF 3.8 CUBIC FEET.[1]

Proportions by Parts			Proportions by Volume			Volume of mortar in terms of percentage of volume of stone.	Average Volume of Rammed Concrete Made from One Barrel Cement — Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	**45%**†	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	**cu. ft.**	cu. ft.	cu. ft.	cu. ft.
1		1	1		3.8	94	5.3	5.5	5.7	6.2	6.7
1		2	1		7.6	51	7.4	7.8	8.2	9.2	10.2
1		3	1		11.4	36		10.0	10.6	12.2	13.6
1		4	1		15.2	29				15.2	17.1
1		5	1		19.0	25				18.2	20.6
1		6	1		22.8	22				21.1	24.0
1		7	1		26.6	20					27.5
1		8	1		30.4	19					31.0
1		9	1		34.2	18					34.4
1		10	1		38.0	17					37.9
1		11	1		41.8	16					41.4
1		12	1		45.5	15					44.8
1	1	1½	1	3.8	5.7	90	8.5	8.8	9.1	9.7	10.3
1	1	2	1	3.8	7.6	75	9.5	9.9	10.3	11.1	11.9
1	1	2½	1	3.8	9.5	61	10.5	11.0	11.5	12.6	13.6
1	1	3	1	3.8	11.4	51	11.5	12.2	12.8	14.0	15.2
1	1½	2	1	5.7	7.6	93	10.8	11.3	11.7	12.5	13.3
1	1½	2½	1	5.7	9.5	76	11.9	12.4	12.9	13.9	15.0
1	1½	3	1	5.7	11.4	64	12.9	13.5	14.1	15.4	16.6
1	1½	3½	1	5.7	13.3	55	13.9	14.6	15.4	16.8	18.2
1	1½	4	1	5.7	15.2	49	15.0	15.8	16.6	18.2	19.9
1	1½	4½	1	5.7	17.1	44	16.0	16.9	17.8	19.7	21.5
1	1½	5	1	5.7	19.0	40	17.0	18.0	19.1	21.1	23.2
1	2	3	1	7.6	11.4	75	14.3	14.9	15.5	16.7	18.0
1	2	3½	1	7.6	13.3	65	15.3	16.0	16.8	18.2	19.6
1	2	4	1	7.6	15.2	57	16.3	17.2	18.0	19.6	21.3
1	2	4½	1	7.6	17.1	51	17.4	18.3	19.2	21.0	22.9
1	2	5	1	7.6	19.0	47	18.4	19.4	20.4	22.5	24.5
1	2	5½	1	7.6	20.9	43	19.4	20.5	21.7	23.9	26.2
1	2	6	1	7.6	22.8	40	20.4	21.7	22.9	25.4	27.8
1	2½	3	1	9.5	11.4	87	15.7	16.3	16.9	18.1	19.3
1	2½	3½	1	9.5	13.3	75	16.7	17.4	18.1	19.6	21.0
1	2½	4	1	9.5	15.2	66	17.7	18.5	19.3	21.0	22.6
1	2½	4½	1	9.5	17.1	60	18.7	19.6	20.6	22.4	24.3
1	2½	5	1	9.5	19.0	54	19.8	20.8	21.8	23.9	25.9
1	2½	5½	1	9.5	20.9	49	20.8	21.9	23.0	25.3	27.6
1	2½	6	1	9.5	22.8	46	21.8	23.0	24.3	26.7	29.2
1	2½	6½	1	9.5	24.7	42	22.8	24.2	25.5	28.2	30.8
1	2½	7	1	9.5	26.6	40	23.9	25.3	26.7	29.6	32.5
1	3	4	1	11.4	15.2	76	19.1	19.9	20.7	22.4	24.0
1	3	4½	1	11.4	17.1	68	20.1	21.0	21.9	23.8	25.6
1	3	5	1	11.4	19.0	61	21.1	22.1	23.2	25.2	27.2
1	3	5½	1	11.4	20.9	50	22.1	23.3	24.4	26.7	28.9
1	3	6	1	11.4	22.8	52	23.2	24.4	25.6	28.1	30.6
1	3	6½	1	11.4	24.7	48	24.2	25.5	26.9	29.5	32.2
1	3	7	1	11.4	26.6	45	25.2	26.7	28.1	31.0	33.8
1	3	7½	1	11.4	28.5	42	26.2	27.8	29.3	32.4	35.5
1	3	8	1	11.4	30.4	40	27.3	28.9	30.6	33.8	37.1
1	4	5	1	15.2	19.0	76	23.9	24.9	25.9	28.0	30.0
1	4	6	1	15.2	22.8	64	25.0	27.2	28.4	30.8	33.3
1	4	7	1	15.2	26.6	55	28.0	29.4	30.8	33.7	36.6
1	4	8	1	15.2	30.4	49	30.0	31.7	33.3	36.6	39.9
1	4	9	1	15.2	34.2	44	32.1	33.9	35.8	39.4	43.1
1	4	10	1	15.2	38.0	40	34.1	36.2	38.2	42.3	46.4
1	5	10	1	19.0	38.0	47	36.9	38.9	41.0	45.1	49.2
	6	12	1	22.8	45.5	46	43.7	46.2	48.6	53.6	58.5

Note.— Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.
*Use 50% column for broken stone screened to uniform size.
†***Use 45% column for average conditions*** and for broken stone with dust screened out.
‡Use 40% column for gravel or mixed stone and gravel.
§Use these columns for scientifically graded mixtures.

[1] From "Concrete, Plain and Reinforced," Taylor and Thompson. Copyright, 1905 and 1907, by Frederick W. Taylor. John Wiley & Sons, Inc., New York.

TABLE LVIII.—VOLUME OF CONCRETE BASED ON A BARREL OF 4 CUBIC FEET.[1]

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUME			Volume of mortar in terms of percentage of volume of stone	AVERAGE VOLUME OF RAMMED CONCRETE MADE FROM ONE BARREL CEMENT — Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	45%†	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	cu. ft.	cu. ft.	cu. ft.	cu. ft.
1		1	1		4	89	5.4	5.6	5.8	6.4	6.9
1		2	1		8	49	7.6	8.0	8.4	9.5	10.5
1		3	1		12	35		10.4	11.0	12.7	14.2
1		4	1		16	28				15.8	17.8
1		5	1		20	24				18.9	21.5
1		6	1		24	22				22.1	25.1
1		7	1		28	20					28.8
1		8	1		32	18					32.4
1		9	1		36	17					36.1
1		10	1		40	16					39.7
1		11	1		44	15					43.4
1		12	1		48	15					47.0
1	1	1½	1	4	6	96	8.8	9.1	9.4	10.0	10.7
1	1	2	1	4	8	73	9.8	10.3	10.7	11.6	12.4
1	1	2½	1	4	10	59	10.9	11.5	12.0	13.1	14.2
1	1	3	1	4	12	50	12.0	12.7	13.3	14.6	15.0
1	1½	2	1	6	8	92	11.3	11.7	12.2	13.0	13.9
1	1½	2½	1	6	10	74	12.4	12.9	13.5	14.5	15.6
1	1½	3	1	6	12	62	13.5	14.1	14.8	16.0	17.3
1	1½	3½	1	6	14	54	14.5	15.3	16.0	17.6	19.1
1	1½	4	1	6	16	48	15.0	16.5	17.3	19.1	20.8
1	1½	4½	1	6	18	43	16.7	17.7	18.6	20.6	22.5
1	1½	5	1	6	20	30	17.8	18.9	19.9	22.1	24.3
1	2	3	1	8	12	74	14.9	15.6	16.2	17.5	18.8
1	2	3½	1	8	14	64	16.0	16.7	17.5	19.0	20.5
1	2	4	1	8	16	56	17.1	17.9	18.8	20.5	22.3
1	2	4½	1	8	18	51	18.1	19.1	20.1	22.0	23.9
1	2	5	1	8	20	46	19.2	20.3	21.4	23.5	25.7
1	2	5½	1	8	22	42	20.3	21.5	22.7	25.1	27.4
1	2	6	1	8	24	39	21.4	22.7	24.0	26.6	29.2
1	2½	3	1	10	12	86	16.3	17.0	17.6	18.9	20.2
1	2½	3½	1	10	14	75	17.4	18.2	18.9	20.5	22.0
1	2½	4	1	10	16	66	18.5	19.4	20.2	21.9	23.7
1	2½	4½	1	10	18	50	19.6	20.6	21.5	23.5	25.4
1	2½	5	1	10	20	54	20.7	21.8	22.8	25.0	27.2
1	2½	5½	1	10	22	49	21.8	22.9	24.1	26.5	28.9
1	2½	6	1	10	24	45	22.8	24.1	25.4	28.0	30.6
1	2½	6½	1	10	26	42	23.9	25.3	26.7	29.5	32.3
1	2½	7	1	10	28	39	25.0	26.5	28.0	31.0	34.0
1	3	4	1	12	16	75	20.0	20.8	21.7	23.4	25.1
1	3	4½	1	12	18	67	21.0	22.0	23.0	24.9	26.8
1	3	5	1	12	20	60	22.1	23.2	24.3	26.4	28.6
1	3	5½	1	12	22	55	23.2	24.4	25.6	28.0	30.3
1	3	6	1	12	24	50	24.3	25.6	26.9	29.5	32.1
1	3	6½	1	12	26	48	25.4	26.8	28.2	31.0	33.8
1	3	7	1	12	28	44	26.4	27.9	29.4	32.5	35.5
1	3	7½	1	12	30	42	27.5	29.1	30.8	34.0	37.2
1	3	8	1	12	32	39	28.6	30.3	32.0	35.5	39.0
1	4	5	1	16	20	75	25.0	26.1	27.2	29.3	31.5
1	4	6	1	16	24	63	27.2	28.5	29.8	32.4	35.0
1	4	7	1	16	28	55	29.3	30.8	32.4	35.4	38.4
1	4	8	1	16	32	48	31.5	33.2	34.9	38.4	41.0
1	4	9	1	16	36	45	33.6	35.6	37.5	41.4	45.3
1	4	10	1	16	40	40	35.8	38.0	40.1	44.4	48.8
1	5	10	1	20	40	47	38.7	40.9	43.0	47.3	51.7
1	6	12	1	24	48	46	45.9	48.5	51.1	56.3	61.4

NOTE. — Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.
*Use 50% column for broken stone screened to uniform size.
†Use 45% column for average conditions and for broken stone with dust screened out.
‡Use 40% column for gravel or mixed stone and gravel.
§Use these columns for scientifically graded mixtures.

tities of cement, sand and broken stone to order for a given number of yards of concrete of certain specified proportions. The most convenient way of determining these quantities is to prepare tables from which the amounts required for a yard of concrete of any proportion may be taken, using only such proportions as have the voids in the stone entirely or more than filled.

"Many writers on this subject have assumed that every concrete, no matter what the proportions may be, has simply a yard of broken stone, without regard to the fact that there is more or less mortar in different proportions. In making up a table of this sort the speaker used the proportions of mortar given in Baker's 'Masonry Construction,' but found that this did not agree with actual practice. These tables in 'Masonry Construction' were made up from theoretical considerations, and it is understood that they are to be revised in a new edition. [This has since been done.]

"The table was made up some years ago by comparison with actual cases, and has been found to give very satisfactory results in practice. The column for broken stone with 0.4 voids will represent closely ordinary broken limestone which chips up more than a harder stone and consequently has less voids; while the column with 0.5 voids represents trap rock, the break in which is more angular. It will be readily seen from the table for Portland cement concrete that certain proportions will not have the voids filled and should not be used.

"For example, with 1 to 3 mortar and with stone having 0.4 voids all the concrete given will have the voids filled, or more than filled, while with 0.5 voids only the 1–3–4 (No. 7) will have the voids filled. The table is made up on the basis of shrinkage under mixing and ramming of about 7 cubic feet, and 3.8 cubic feet of cement to a barrel.

"In a recent paper presented to the society, one of the members stated that concrete was not a fit material for piers of railway bridges, unless a pedestal block of stone was used on which to rest the masonry plates. The speaker would refer that member to the engineers of half a dozen large western roads who use concrete for piers, and allow the bed plates to rest directly upon the concrete. Concrete is certainly better than most stone, except granite, and probably 50 per cent. better than much of the stone commonly used."

The tables (LV, LVI, LVII and LVIII) by Taylor and Thompson in Concrete Plain and Reinforced are the most complete now in use, and are published by permission.

TABLE LIX.—FOWLER'S PROPORTIONS FOR PORTLAND CEMENT CONCRETE.

Number.	Proportions.	Barrels Cement.	Cubic Yards. Sand.	Stone. 0.4 Voids.	Stone, 0.5 Voids.
1	1-2-3	1.77	0.51	0.87	1.05
2	1-2-3½	1.68	0.49	0.91	1.10
3	1-2-4	1.59	0.47	0.95	1.15
4	1-2-4½	1.48	0.44	1.00	1.20
5	1-2-5	1.39	0.42	1.04	1.26
6	1-2-5½	1.30	0.40	1.08	1.30
7	1-3-4	1.30	0.57	0.83	1.00
8	1-3-4½	1.22	0.54	0.89	1.06
9	1-3-5	1.16	0.52	0.92	1.11
10	1-3-5½	1.09	0.50	0.97	1.16
11	1-3-6	1.04	0.48	1.00	1.20
12	1-4-6	1.00	0.55	0.91	1.09
13	1-4-6½	0.96	0.53	0.94	1.13
14	1-4-7	0.92	0.51	0.97	1.17
15	1-4-7½	0.88	0.49	1.00	1.21
16	1-4-8	0.83	0.47	1.03	1.25

About 34 cubic feet loose = 1 cubic yard rammed.
Labor = ½ to 2 cubic yards per man per day.

Copy of Report of the Operations of the Engineering Department of the District of Columbia, under the direction of Major Charles F. Powell, Corps of Engineers, U. S. A., Engineer Commissioner of District of Columbia, year ending June 30, 1896. (Page 194).

TABLE LX.—TENSILE STRENGTH: 3 PARTS QUARTZ, 1 PART CEMENT.

	7 Days.	1 Month.	2 Months.	3 Months.	4 Months.	5 Months.	6 Months.	12 Months.
Atlas	321	441	441	510	519	525	538	546
Alsen	188	310	290	328	385	380	390	366
Dyckerhoff	164	175	192	236	257	293	298	323
Hanover	205	244	251	277	301	315	315	354
Alpha	105	182	310	309	310	295	327	350
Hemmoor	159	203	286	301	323	329	314	347
Giant	230	275	275	267	296	329	325	327
Porta	181	257	305	319	315	322	343	329
Egypt	159	205	255	240	285	301	341	394
Henry	159	188	229	277	300	320	319	332
Mannheimer	193	226	306	329	335	323	343	336
Saylor's	135	156	205	203	254	277	289	270

Summary of cement tests for tensile strength and fineness made by the Seattle City Engineer's Office to this date, March 10, 1904.

TABLE LXI.—MIXTURE: 1 VOLUME OF CEMENT TO 1 VOLUME OF SAND.

Brand.	No. Briquettes.	Tensile Strength.						Fineness. On No. 100 Sieve, 10,000 Meshes to Sq. Inch.
		8 Days.	31 Days.	3 Months.	6 Months.	1 Year.	2 Years.	
Alsen	6	415	480	547	560	634	664	87%
Condor	6	319	458	555	606	641	703	87.5
Dykerhoff	7	443	482	561	605	691	...	90
Flying Cask	12	472	580	604	...	...	...	90
Fortification	6	309	361	477	479	...	...	93
Germania	6	308	412	484	510	606	...	92
Golden Gate	17	297	424	517	537	...	...	84.8
Harcourt	6	207	300	341	366	424	466	87
Heidelberg	6	153	240	284	363	393	424	88.5
Hemmoor	6	306	391	446	504	557	...	87
Hercules	6	175	289	354	384	448	...	85.3
Josson	6	259	369	425	475	530	584	88
K. B. & S.	6	273	286	424	...	...	...	87.5
Mannheimer	6	341	401	470	519	507	585	88
Red Castle	6	215	261	334	402	488	526	87
Rudersdorf	6	331	425	481	545	591	...	91
Standard	18	357	428	573	587	...	...	94.5
Teutonia	5	330	400	433	475	508	...	85
Trowel	6	178	252	348	404	475	...	85
Utah	6	320	461	475	556	592	603	85

TABLE LXII.—CEMENT TESTS, CITY OF SEATTLE, 1913

(Pacific Coast Cements)

Brand.	Fineness 200 Sieve.	Strength, 1 Cement and 3 Sand.			
		7 days.	28 days.	3 mos.	6 mos.
Superior... (Washington)	89.4	314	425	512	655
Washington "	87.8	300	398	492	503
Olympic "	92.3	430	520		
International "	85.0	380	424		
Inland "	84.0	296	373		
Golden Gate (California)	87.0	340	420	534	536
Blue Cross "	89.0	360	440	586	595
Riverside "	84.0	280	360		
Mt. Diablo "	90.0	325	420	577	560
Standard "	84.0	282	365		

TABLE LXIII.—CAPACITIES, WEIGHTS, DIMENSIONS RANSOME MIXERS.

Model, Number of Mixer.	*59	*60	*61	*62	63	64	66	67
Size of batch, cu. ft. loose material	8	10	18	24	40	80	21	28
Capacity per hour, cubic yards	7	10	18	24	40	80	21	28
H.P. of engine	6	6	8	9	20	35	8	12
H.P. of boiler	7	9	12	14	27	50	12	15
R.P.M. of drum	21	21	20	17	16	12	20	17
R.P.M. of countershaft	184	184	175	165	165	116	165	165
Diameter and length of drum	42×35½	48×36	54×40	60×46	66×54	84×70	54×40	60×46
Thickness of drum plates	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{3}{16}$
Thickness of drum blades	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{4}$
Diameter of roller shafts	$1\frac{7}{16}$	$1\frac{15}{16}$	$1\frac{15}{16}$	$1\frac{15}{16}$	$2\frac{7}{16}$	$2\frac{15}{16}$	$1\frac{15}{16}$	$1\frac{15}{16}$
Diameter of countershaft	$1\frac{15}{16}$	$1\frac{15}{16}$	$2\frac{7}{16}$	$2\frac{7}{16}$	$2\frac{15}{16}$	$3\frac{7}{16}$	$2\frac{7}{16}$	$2\frac{7}{16}$
Thickness of traction rings	$\frac{5}{8}$	1	$1\frac{1}{8}$	$1\frac{3}{8}$	$1\frac{5}{8}$	2	$1\frac{1}{8}$	$1\frac{3}{8}$
Pitch of teeth of gear ring	$1\frac{1}{4}$	$1\frac{1}{2}$	2	2	2	$2\frac{1}{2}$	2	2
Width of face of gear ring	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{1}{2}$	6	$2\frac{1}{2}$	$2\frac{1}{2}$
Length of journal	6	6	6	6	7	8	6	6
H.P. of gasoline engine	5	6	10	10	....	....	10	12
H.P. of electric motor	5	7½	10	15	30	50	10	15
Net weight of mixer on skids	2,400	3,100	3,800	5,000	7,200	14,700	3,900	5,300
Boxed weight of mixer on skids	3,100	3,900	4,700	6,300	8,600	16,100	4,800	6,600
Cubic feet of mixer on skids	140	180	220	310	400	842	220	310
Net weight mixer and steam engine on skids	3,400	4,300	5,500	6,650	9,900	20,000	5,600	7,300
Boxed weight mixer and steam engine on skids	4,300	5,250	6,600	8,300	11,300	22,500	6,700	9,000
Cubic feet mixer and steam engine on skids	175	225	250	390	475	890	250	395
Net weight mixer, engine and boiler on skids	5,100	6,100	7,800	9,000	15,000	26,000	7,900	9,700
Boxed weight, mixer, engine and boiler on skids	6,400	7,400	9,150	11,250	17,100	31,000	9,250	11,900
Cubic feet, mixer, engine and boiler on skids	275	325	390	560	695	1,160	390	565
Net weight mixer and gas engine on skids	3,900	5,800	7,200	8,150	....	....	7,200	8,400
Boxed weight mixer and gas engine on skids	4,800	6,800	8,300	9,700	....	....	8,400	10,000
Cubic feet mixer and gas engine on skids	240	270	330	480	....	....	330	480
Net weight mixer and motor on skids	3,800	4,900	6,400	6,800	10,000	19,500	6,500	7,100
Boxed weight of mixer and motor on skids	4,700	5,850	7,500	8,000	11,400	22,400	7,600	8,300
Cubic feet mixer and motor on skids	175	220	250	360	435	840	250	360
Extra weight of trucks	590	590	1,250	1,250	....	....	1,250	1,250

Shipping weights and dimensions approximate, and subject to revision.

NOTE.—Models 60, 61, and 62 are made with the gear attached to one of the traction rings, and have solid babbitted journals instead of bronze brushed as furnished on models 63, 64, 66, and 67.

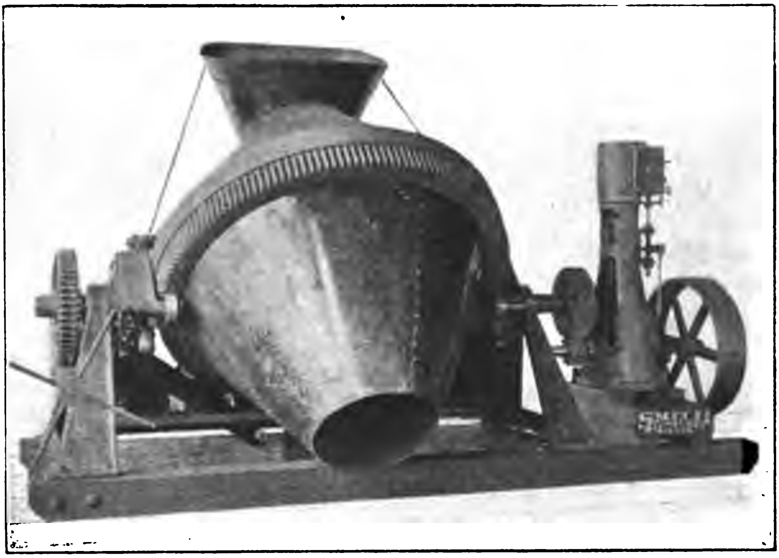


Fig. 357.—Smith Concrete-mixer.

Fig. 358.—Green Lake Sand and Gravel Bunkers.

For machine mixing the new Ransome mixer, or the Smith mixer (Fig. 357) are considered by the author to be among the best for turning out thoroughly and uniformly mixed concrete.

The wonderful increase in the use of concrete since 1900 has led to the study of cheapening its cost in every possible way. The placing of 20,000 yards in the Green Lake reservoirs at Seattle by the author was accomplished by using the mixing plant shown in Fig. 358.

The sand and gravel was towed on scows a distance of forty miles, by a tug and scow outfit described in Chapter XIX, to an unloading dock at the waterfront, where it was transferred to

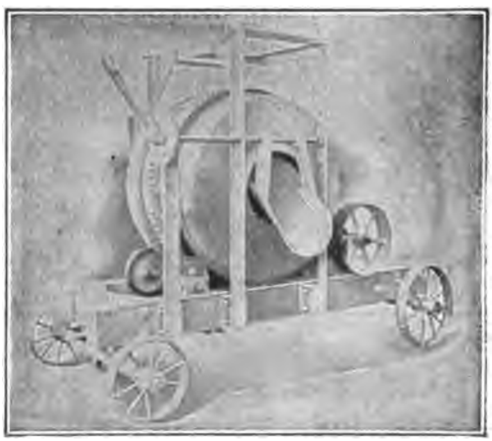


FIG. 359.—RANSOME CONCRETE MIXER.

Rogers ballast cars by a 1-yard clam-shell bucket, operated by a double 7×12 hoist engine, the hoisting lines being used to swing the boom of the derrick as described in Chapter XVI.

The cars were switched, about 10 or 12 each day, by steam railroad a distance of about three miles, to an electric transfer point where electric locomotives handled them to the bunkers, over several miles of street railway and over one and one-half miles of standard-gage track constructed for this particular work; several blocks of the line being on an 8 per cent. grade.

The Ransome mixer without power and trucks is shown in Fig. 359 and with hoppers and gasoline engines in Fig. 360 and Fig. 361.

The drum is a cylinder of rolled steel plate, with heavy, wide-face traction surfaces. To the inner periphery are bolted the Ransome

mixing blades. These blades carry the materials up to the highest point of the drum, whence they are thrown down on the mass in the lower part of the drum, producing a mixing action which is a

FIG. 360.—RANSOME GASOLINE-DRIVEN OUTFIT WITH FIXED BATCH HOPPER.

FIG. 361.—RANSOME GASOLINE-DRIVEN OUTFIT WITH PIVOT HOPPER.

combination of rolling and grinding contact. Water clearances, a patented Ransome feature, are provided beneath and at the ends of the blades. This insures quick and thorough cleaning.

The gear ring is made of cast gray iron, with teeth of two-inch pitch. The journals are of the solid babbitted type.

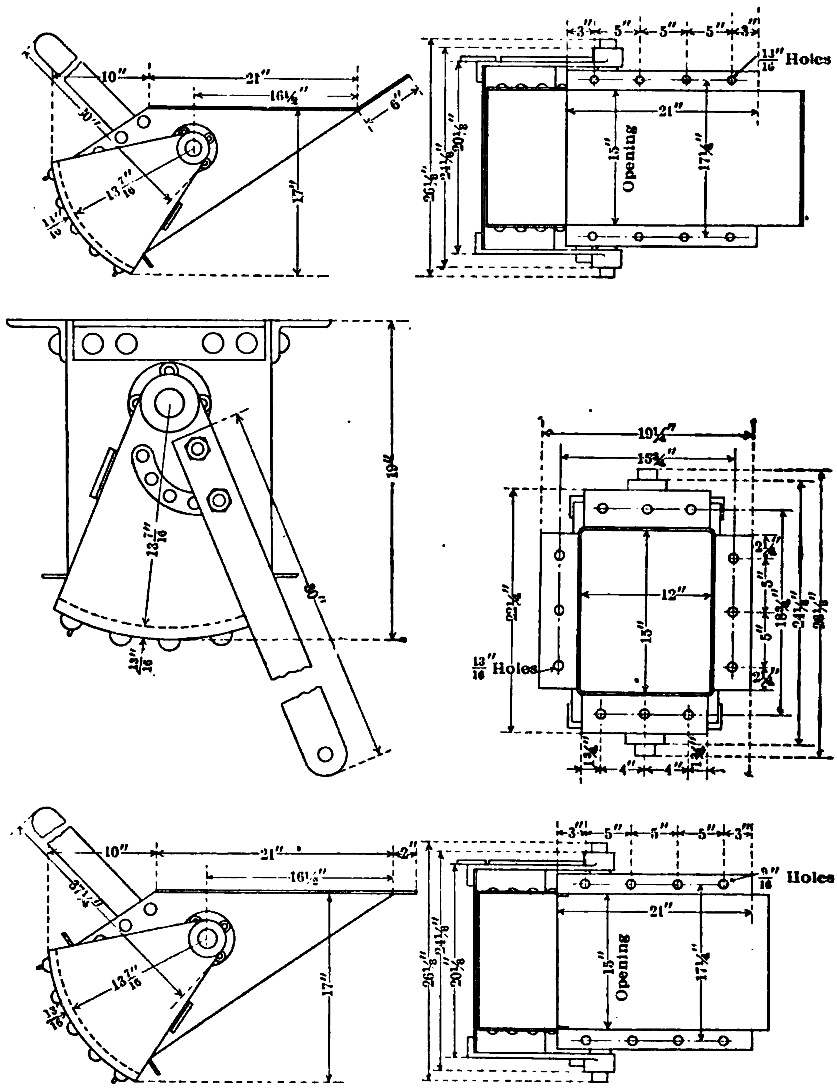


FIG. 362.—RANSOME BIN GATES.

The cars were run on top of the bunkers, dumping into the sand bunkers from one track and into the gravel bunkers from the other. The sand and gravel were discharged from the bunkers through a

gate similar to Fig. 362 (*a*), into a wooden measuring hopper on the mixer, similar to the one shown in Fig. 360. The mixer used for the 1–3–5 concrete was a 30-cubic-foot Ransome, and the mixer used for the 1–$1\frac{1}{2}$ cement mortar top was a 20-cubic-foot Ransome.

Fig. 363.—Concrete Barrow.

Fig. 364.—Ransome Concrete Cart.

The concrete was then discharged into Koppel dump-cars, operating similarly to Fig. 365, and which were pushed by one man over 18-inch gage track with 20-pound rails, to the place where work was in progress. The work on the slopes was carried on by picking up the body of the cars by a bridle from a 10-ton derrick with long

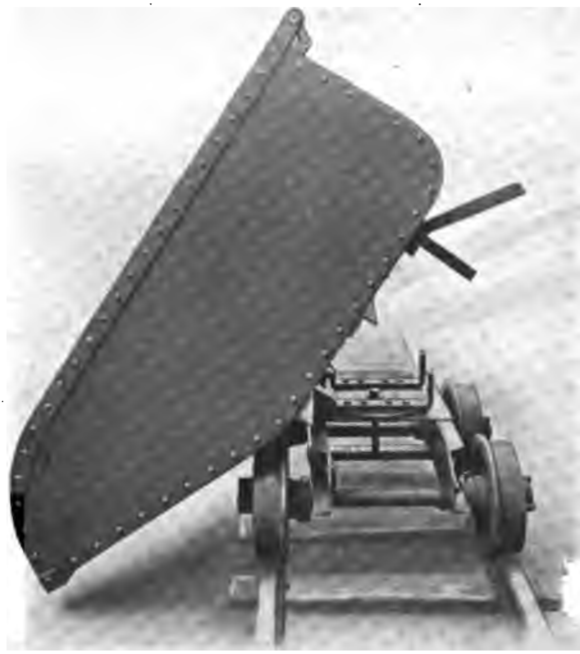


FIG. 365.—RANSOME CONCRETE ROTARY DUMP CAR.

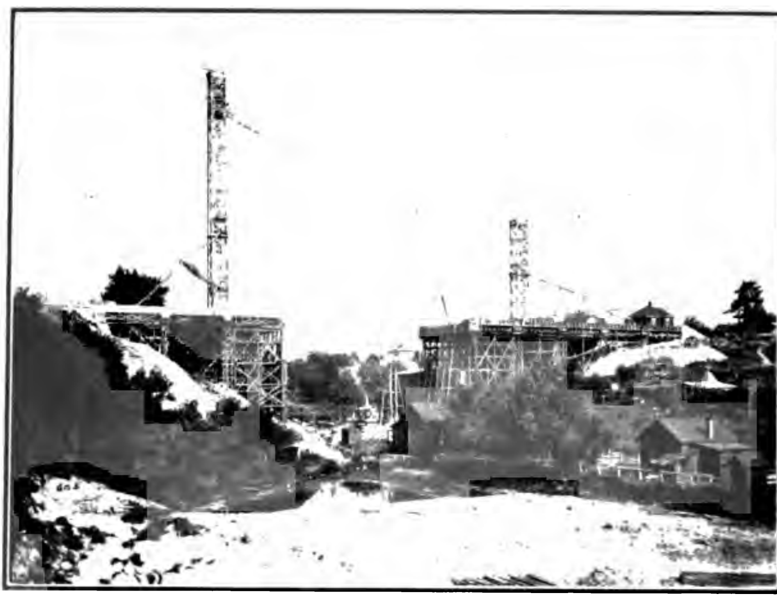


FIG. 366.—CONCRETE SPOUTING PLANT, PORTLAND, ORE.

boom, and they were swung around and dumped into place. The concrete had to be a dryer mix for the slopes, to prevent running, than was used for the bottom. The plant worked very smoothly and handled on this particularly hard kind of work, from 200 to 250 yards per day of eight hours.

FIG. 367.—TOWER 186 FEET HIGH. SPOUTING PLANT, PORTLAND.

The placing of concrete is accomplished on small jobs by using ordinary concrete wheel-barrows (Fig. 363), but an output of about 150 yards per day, requires usually about thirty men, and consequently for work of a medium size, concrete carts (Fig. 364) are used, and often larger carts of a similar type drawn by horses. With

the hand carts the crew can be cut down over one-half, while with the horse carts only about one-third the number of men are needed.

Where the work is strung out in a line, cableways can be used, as described in the preceding pages, but on nearly all work of large size, the spouting system of distributing concrete is now employed. The plant used by the author at the East Twenty-first Street Viaduct in Portland (Fig. 366) consisted of two towers; the tall one (Fig. 367) at the mixer for hoisting the concrete was 186 feet high, while the secondary one for distributing the concrete was 100 feet high.

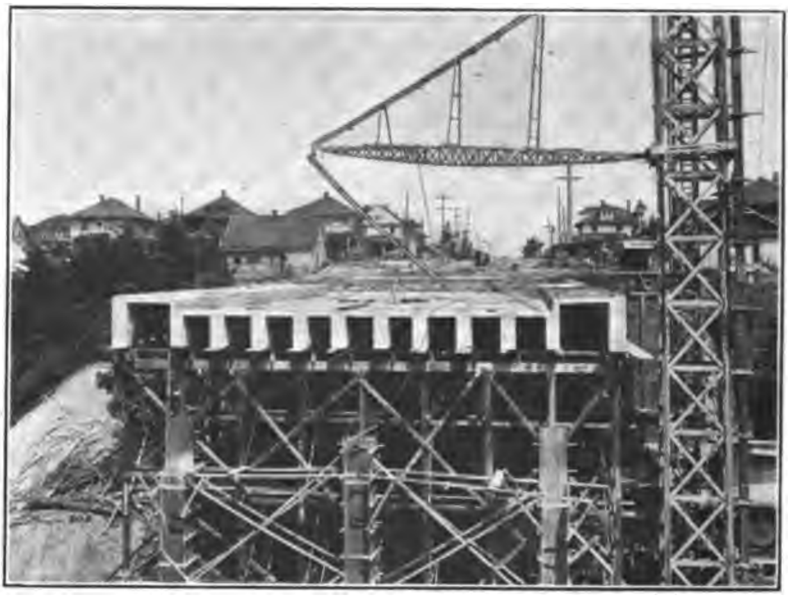


Fig. 368.—Distributing Boom on Spouting Plant, Portland, Ore.

The booms for handling the concrete were of the style shown in Fig. 368.

The towers in this case were of light timber construction, guyed with small wire ropes. Experience on the work indicated that the tall tower properly placed would have handled the work just as well or better, than the two laid out by the patentees. Where such a system is used for a dam or heavy mass of concrete, the tower can be of steel and be buried in the work.

The placing of the concrete in the above case was carried out at about the same cost for less than 4000 yards placed, as would have resulted from the use of cars and tracks. This of course includes the

cost of plant and its erection and removal. With a larger yardage the result would have been very much better, as close to 300 yards could be placed in one day with twelve men. The ordinary cost of such mixing and placing on large work runs from 30 to 40 cents

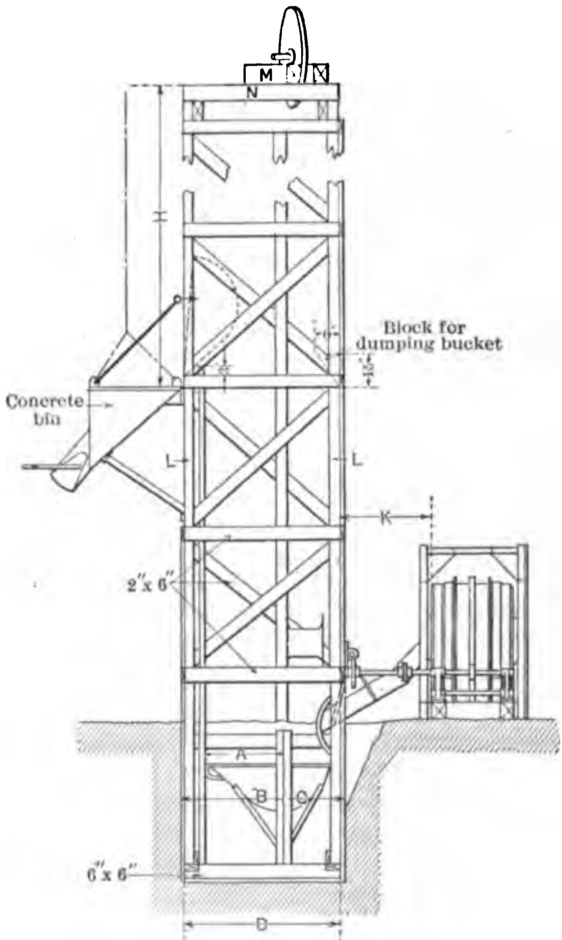


FIG. 369.—SIMPLEST PLANT FOR SPOUTING CONCRETE.

per yard, and the figures given by the makers of such plants are seldom or never realized in practice.

The arrangement and details of such a tower are shown in Fig. 369, and another plant used by the author in placing about 7000 yards of concrete within a 175-foot radius is shown in Fig. 90, the one tower working in a perfectly satisfactory manner.

CHAPTER XXVII

FOUNDATIONS FOR PIERS AND WHARVES*

THE types of construction used in piers and wharves have been a matter of very slow evolution. Those built with foundations of piling and timber and of fills out into the water are the prototypes of the various classes of construction used on the best work at the present time. Structures of this class are of two general types, piers or jetties extending out from the shore into the water, and wharves or quays built along the shore or parallel to it. Many of the European ports have both types, and the same is true of practically all the principal ports of the world. Those constructed on Puget Sound are largely of timber, and at Seattle, one of the greatest ports of the world, both the jetty and quay type are used, although the jetty type is used in the majority of cases. Those at Tacoma, 30 miles farther up the Sound, on the other hand, are practically all of the quay type.

The ordinary piers constructed as landing places for small vessels are very often built with untreated piles and untreated timber. Where the teredo are very active untreated peeled piles will be eaten off in a few weeks or in a few months at the outside. Piles driven with the bark on, where it is not broken, will last from one to two years. An illustration showing a section of such a pile which had been in use for about two years is shown in Chapter XXVIII, consequently, the semi-permanent class of piers are constructed with creosoted piling, which will last approximately the same number of years as there are pounds of creosote per cubic foot. These piles in recent years have nearly all been treated by the boiling process described in Appendix X. The treatment should never be less than 12 pounds per cubic foot for any timber, and for the softer grades it should be about 16 pounds per cubic foot, and in places where teredo are particularly bad, 20 pounds per cubic foot. Douglas fir from the northwestern portion of the United States is a very difficult species of timber to treat, and it is doubtful if more than 16 pounds can be put into it uniformly by any process, the same being true of other very hard-fibered timber.

* See costs of piers in Chapters XXXII and XXXIII.

The bracing of timber piers is ordinarily done by using creosoted brace piles driven at an angle of from 30 to 45 degrees (Fig. 374), depending on the depth of the water, but in some instances creosoted plank for X-bracing is bolted or spiked on with galvanized bolts or spikes. With this exception it is very rarely that any creosoted lumber is used in the construction of timber piers, although the creosoting of the caps and joists or stringers would be a very wise thing to do. The ordinary small pier has the bents located about 12 feet center to center, with the piles in each bent spaced about 10 feet center to center. The caps are usually 12×12 or 12×14, the joists 4×14, and the floor planking 3 inches thick. Such a wharf will cost from 40 to 50 cents per square foot of its size in plan. The heavier piers constructed by using creosoted piling costs ordinarily from 70 to 80 cents per square foot, while those of the heaviest type, such as are later described as having been constructed at the Puget Sound Navy Yard, will cost approximately $1.20 per square foot.

The depreciation, from the author's data, of timber piers in salt water, where neither the piling nor timber are creosoted, must be taken at not less than 12 to 15 per cent. per annum, and where the teredo are very bad this will easily run to 18 or 20 per cent. Where the piles are creosoted the depreciation will be about 8 to 10 per cent., and when in addition the caps and joist are treated the depreciation may be reduced to under 5 per cent.

The comparative value of piers on creosoted piles and of reinforced concrete is indicated by the following figures from San Francisco types, although the loss of business during the time repairs are in progress has not been taken into account and this would make the showing for reinforced concrete much better.

Assuming the cost of a creosoted pile pier at $1 per square foot with a life of twelve years, and that of a reinforced concrete pier at $3.25 per square foot with a life of one hundred years, the comparative annual charges for the two types of construction are approximately as follows:

	Creosoted Pile.	Reinforced Concrete.
Interest at 4½ per cent	$0.0450	$0.1463
Maintenance assumed	0.0500	0.0125
Replacement, 12 years	0.0681	....
Sinking fund, 75 years	0.0022	0.0077
Insurance	0.0060	....
Total per square foot	$0.1719	$0.1665

This would indicate that the annual cost is very nearly the same for the two classes, although in the author's opinion the maintenance charge for the creosoted pile pier is too small and for the concrete pier slightly large. The above figures are taken from a paper by Jerome Newman, M. Am. Soc. C.E., Chief Engineer Board of State Harbor Commissioners, San Francisco.

The piers constructed in 1913 by the Port of Seattle, of which Gen. H. M. Chittenden, M. Am. Soc. C.E., is president, and Paul P. Whitham is chief engineer, are examples of the moderately heavy type of construction on creosoted piling. A plan of two of these wharves on opposite sides of a common slip is shown in Fig. 370, and a section of one of the wharves in Fig. 371.

The improvements under way will cost over $6,000,000 and when completed there will have been invested in the harbor works

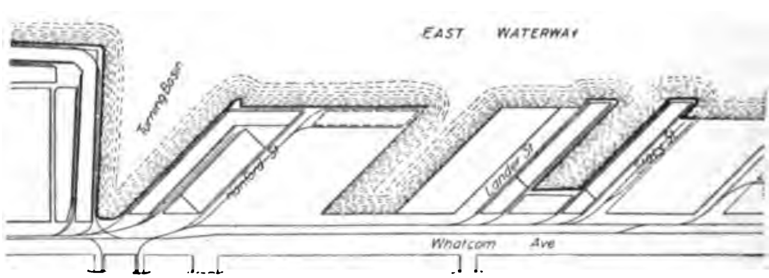


Fig. 370.—Piers and Slip, Seattle.

and piers in Seattle more than $20,000,000, the recent work having been inaugurated by the 1908 Harbor Commission, of which the author was a member.

The piling are creosoted, and spaced from 4 to 5 feet center to center in each bent, with the bents 20 feet apart. The piles are capped with 12×12 timber, the joist are 4×14, and the planking $2\frac{3}{4}$ inches finished thickness. The stringers under the railroad tracks are three 6×16 under each rail. The corner framing and details are as shown in Fig. 372. The spacing of the piles in each bent is somewhat closer, and the spacing of the bents center to center is somewhat wider than usual for this type of pier, the ordinary spacing being about 8 or 10 feet for the piles in each bent, with the bents from 8 to 12 feet center to center, which would, of course, require slightly heavier caps and lighter joists or stringers to support the same load.

The piers constructed at the Puget Sound Navy Yard by the author are practically all of the type shown in Figs. 373, 374 and 375,

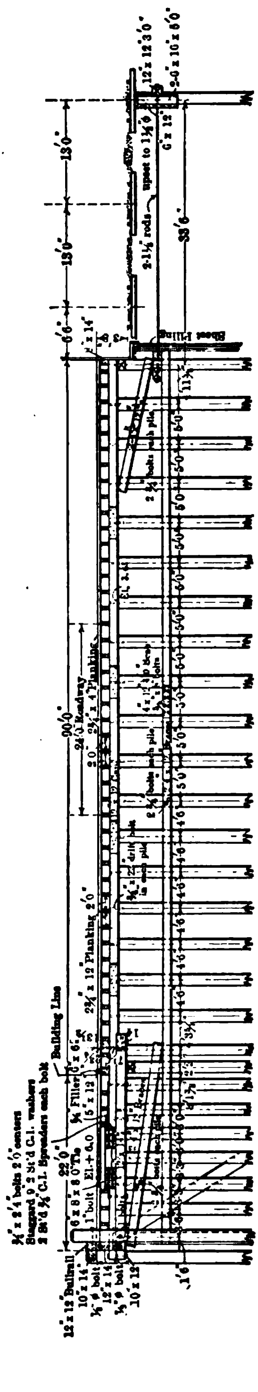


FIG. 371.—CROSS SECTION OF PIER, SEATTLE.

the piles being spaced 8 feet center to center in both directions. The piles for these wharves are usually specified to have 9-inch tops, and not less than 16-inch butts, and to have either 14 or 16 pounds of creosote per cubic foot. The caps are 12×12, the joist 4×14, and the floor planking 4 inches thick. The bracing consists of diagonal brace piles transversely, and 4×12 creosoted X-bracing longitudinally, where such bracing is used. All of the fastenings for the Government work are galvanized, and of the sizes and lengths shown in the illustrations. The bottom at this Navy Yard is very hard, and except for a few feet of soft gravel on top is mostly cemented gravel, so that sometimes the piles simply sniped off to a rough point with an axe can be driven to a depth of 2 or 3 feet into the hard stratum, but very often cast points have to be employed to get any penetration of value. The triangular point shown in Fig. 57 is one recently used by the author on some of this work, and it was very satisfactory. Where the piles are over-driven, they broom up badly, and take on the appearance of the piles shown in Fig. 54, or else buckle up and sometimes break off. This has resulted in the use of concrete wharves similar to the one described later on in this chapter. The attempt was made and carried out with considerable success on one of the pile piers constructed at the Navy Yard to bore a hole for each pile, and the pile was set in the hole and driven to the penetration required, which was 10 feet. This method, however, proved so slow and prohibitive in cost that it has been abandoned.

Probably the finest piers in the United States are those constructed by the Department of Docks in New York City. The nine new piers comprising the Chelsea improvement are partly shown in Fig. 376. These piers, owing to the fact that teredo are very seldom found in the Hudson River, are built with untreated piles and also timber decks. These decks, in many cases, carry a pavement. For the nine piers above mentioned the sheds or warehouses are of two-story, steel and fireproof construction, which have cost approxi-

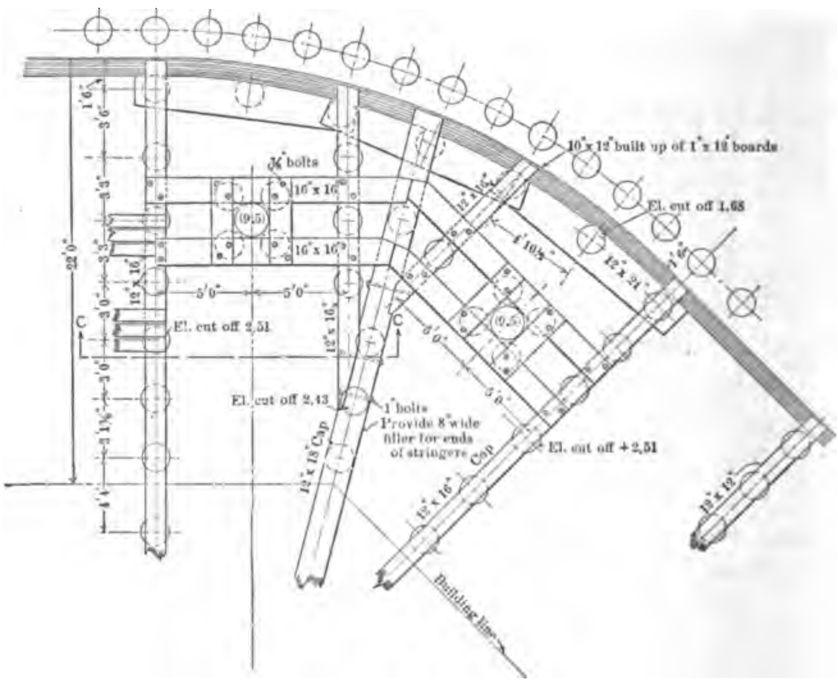
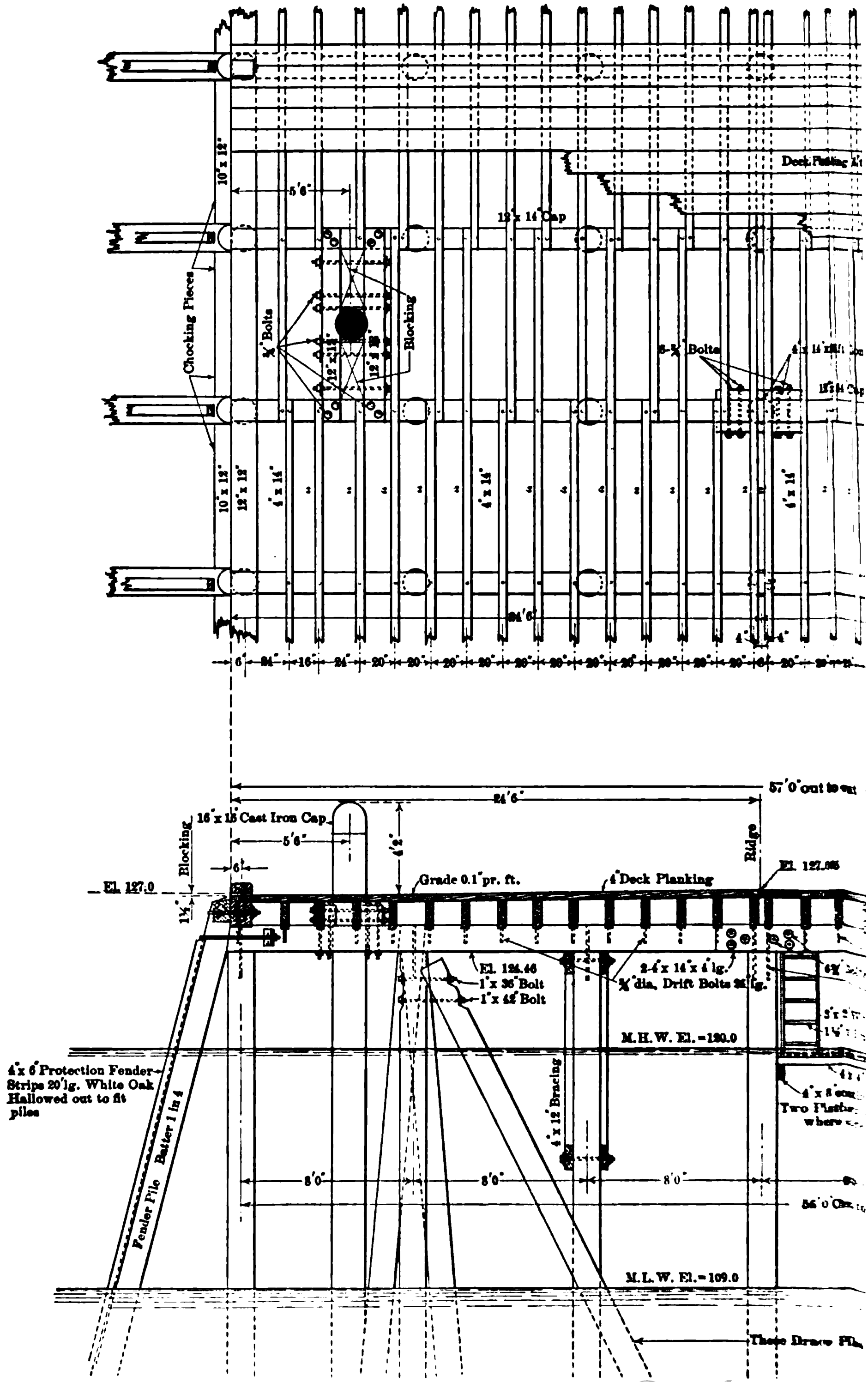


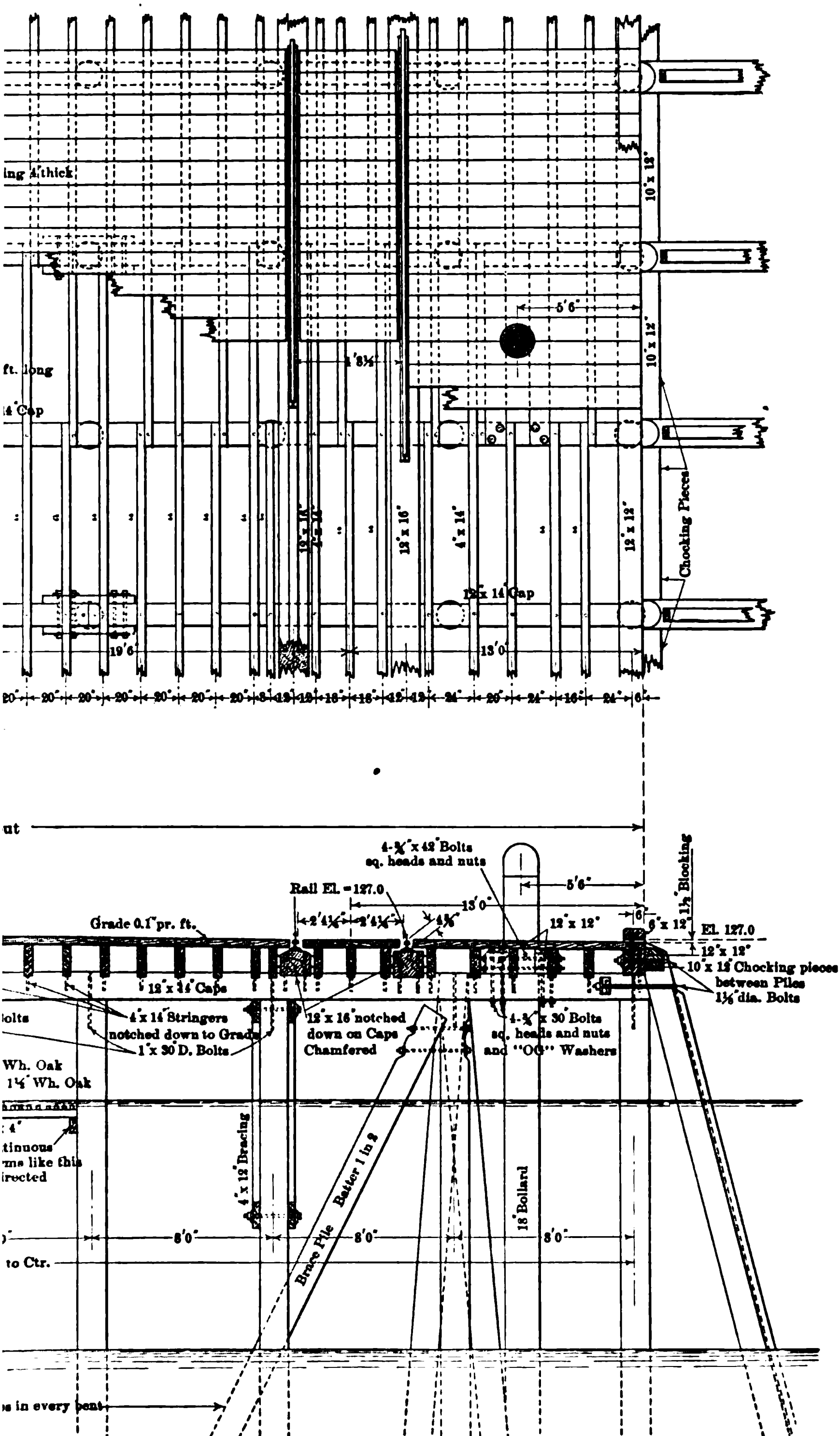
FIG. 372.—DETAIL OF SEATTLE PIERS.

mately 6 cents per cubic foot. The latest type of docks constructed in New York have had a reinforced concrete deck employed, Fig. 377, which also shows the details of construction. The cost of these piers has been slightly less than 90 cents per square foot without the asphalt surfacing, and slightly less than $1 per square foot with asphalt deck. The construction is described in detail in a paper by Charles W. Stanniford, M. Am. Soc. C.E., Chief Engineer, Department of Docks, New York, in the American Society Transactions for May, 1913. The following tabulation gives the data in regard



CROSS

FIG. 374.—PIER AT NAVY



ECTION

'ARD, PLAN AND SECTION.

(*To face page* 547.)

to the repairs required on the old type of wooden deck pier, costing from $1 to $1.15 per square foot.

Description.	Per cent of Total Original Cost.	Renewal Required.
Sheathing	12	Every 6 years.
Backing log	1.8	Every 8 years.
Fender chocks, including vertical sheathing	4	Every 10 years.
Fender piles	4.7	Every 12 years.
Decking	11.3	Every 15 years.
Bracing	7.1	50% in every 30 years.
Rangers and caps	24.4	50% in every 20 years.
Piles	34.7	33⅓% every 20 years. *

* Above M.L.W. only.

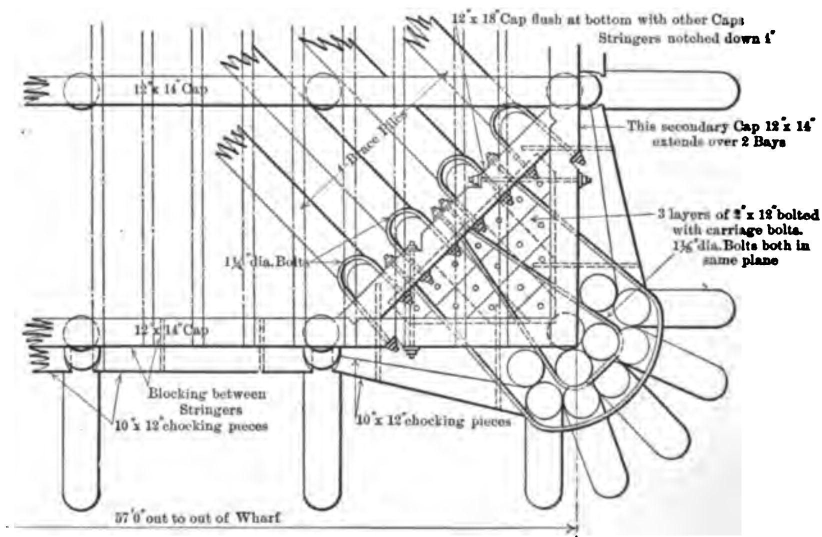
FIG. 373.—CREOSOTED PILE PIER, PUGET SOUND NAVY YARD.

Concrete Deck Pier.—Cost of construction, Thirty-first Street Pier, South Brooklyn, no asphalt surface, \$0.87 per square foot.

Cost of construction, Thirty-third Street Pier, South Brooklyn, with asphalt surface, \$0.97 per square feet.

The following conclusions are drawn from the experience already had with these piers:

" Economy being a prime factor in its construction, it was decided to try out the concrete deck surface for wear and tear of heavy team traffic, and the earlier decks, therefore, were finished with a smooth mortar surface to receive this traffic. Two years of experimenting



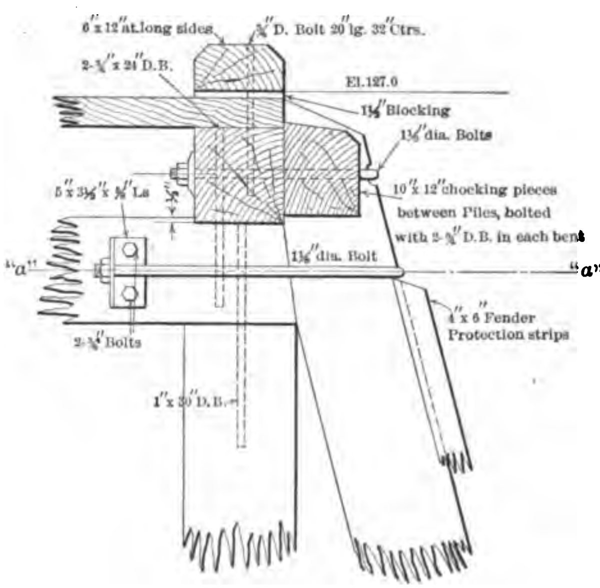
Note: Fender strips not shown

PLAN AT CORNER OF HEAD

FIG. 375.—PIER AT NAVY YARD, CORNER DETAIL.

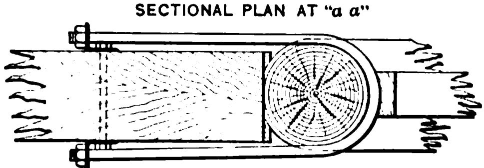
on these lines determined the fact that though the concrete surface was admirably adapted to light traffic, cargo handling by hand or motor trucks, etc., it could not stand the concentration of heavy team traffic confined within narrow lanes located generally in the center of the pier. The grinding and turning of heavily laden trucks inside these narrow lanes or zones gradually caused surface rupture of the top coat of mortar. It was decided, therefore, to place an asphalt wearing surface on the deck, and this has proven very effective.

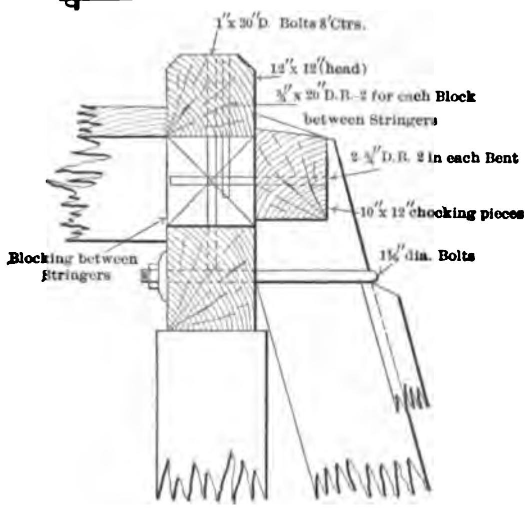
" The piers at the foot of Thirty-first and Thirty-third Streets, South Brooklyn, have been in service for about three years. No



At long sides

SECTIONAL PLAN AT "*a a*"





At head

DETAILS OF BOLTING

FIG. 375*a*.—NAVY PIER DETAILS.

signs of cracking or other imperfections have appeared, and the piers, as a whole, are a complete success.

" For the modern type of concrete deck pier, the cost of maintaining the fender system is about the same as that for the wooden pier; deck sheathing repairs are practically eliminated, except such minor asphalt patching as may be required, and can be considered negligible in a good asphalt deck under cover; the deck plank is eliminated; the life of the ranger and cap system is prolonged by the protection from moisture given by the impervious concrete deck, and the cost of maintenance and repairs, therefore, is reduced to a minimum.

FIG. 376.—CHELSEA PIERS, NEW YORK CITY.

" From the foregoing it will be observed that the problem which confronted the department was the elimination of the timber deck and deck-supporting structure of the wooden pier, by the substitution therefor of some permanent form of construction meeting the following requirements:

" (*a*) Economy in cost of construction and maintenance, the unit cost to be such as to produce or make possible a remunerative return on the capital invested.

" (*b*) The construction to be of such character as to be readily extended, reconstructed, remodeled, or, if necessary, entirely removed, as more intensive development of the area occupied by the pier or system of piers might be made necessary by the growth of commerce and shipping.

" From what has been stated the following conclusions may be deduced:

" *First.* Admitting that timber piles and foundation work are generally permanent below the mean low-water line in New York Harbor, the Department of Docks and Ferries has met the requirements of the problem by producing piers having the following characteristics:

" (*a*) The deck is absolutely permanent;

" (*b*) The substructure, above mean low water, is easily and cheaply repaired and maintained;

" (*c*) The supporting part, below the water line, is permanent; and

" (*d*) The resulting structure is such that it can be readily extended, reconstructed, or, if necessary, entirely removed at a cost not prohibitive, as would be the case, for example, with most types of reinforced concrete deck-supporting structures.

" *Second.* That the department has produced permanent parts in the structures where these are essential. No attempt was made to obtain absolute permanency above low water, in the structure supporting the deck, for the reason that,

" *Third.* This portion of the structure, the caps, piles, braces, etc., protected as they are from saturation by urine and other objectionable fluids by the concrete and asphalt deck forming a protecting roof, can be maintained in good condition at a very low cost.

" *Fourth.* The type of structure produced, approximating permanency, is now being built by the Department at a first cost no greater than that of the former type of wooden pier throughout, and the cost of repairs and maintenance of the deck structure is almost entirely eliminated."

The author, however, cannot agree with the conclusions that a permanent deck should be placed on untreated piling, nor, in his estimation, should any untreated timber be employed in the construction, but all piling and timber be creosoted with not less than 12 pounds per cu.-ft., to at least properly protect it from decay. Should this seem to be out of the question on account of the cost, it would certainly be a wise precaution to apply hot carbolineum to the upper ends of the piling and to all of the timber work. It would seem possible, however, to construct at least that portion of the piers underneath the warehouses with concrete piling, as these are easily made and handled up to 80 or 90 feet.

Some molded, corrugated concrete piles 80 feet in length, Fig. 378, were recently used by the author in the design and construction of a pier for a bascule bridge. These piles were picked up with tackle on the boom of a clam-shell dredge by lashing timbers to them and using a bridle to distribute the weight. After up-ending them they were set in the leads of the pile-driver and readily driven with a cushion cap and by jetting. The experience showed that in large

FIG. 378.—DRIVING CONCRETE PILES. DREDGE PLACING 80-FT. PILE IN LEADS.

quantities about four piles could be driven per day. These piles had extra reinforcing throughout the center half of the piles lengthwise to take care of the bending stresses while they were being handled. Most of the piles were allowed to set thirty days, but a few were driven with only two weeks set, by lashing on extra timbers to pick them up. A reinforced concrete deck was built on the piles for the bridge seat.

The socket concrete pile (Fig. 379) is a somewhat new departure, but could undoubtedly be used in a great many cases where the bottom is soft, and the piles if all of concrete would have to be long and very expensive; with the socket piles resting on wooden

FIG. 378a.—CONCRETE PILES 80 FEET, READY TO POUR.

piles beneath the surface of the mud, the construction would be greatly facilitated, and the cost of the work very greatly reduced.

"The use of pre-cast piles for permanent construction at a reasonable cost is an ideal type. Every pile is known to be perfect and cured before being placed. It is immune from decay, fire, or

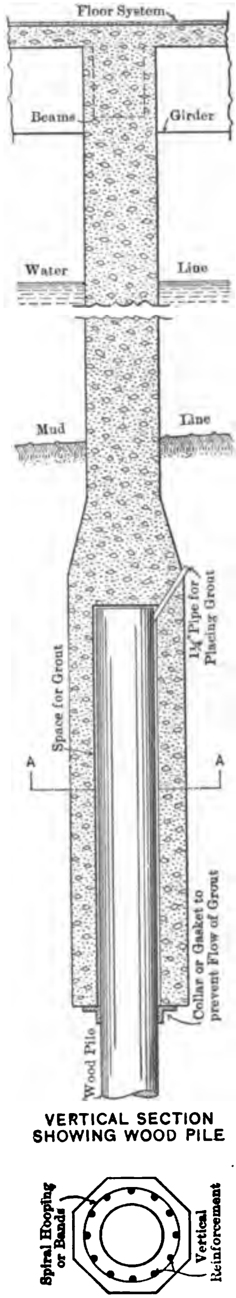


FIG. 379.—SOCKET CONCRETE PILE.

marine-insect attack, and will carry a wood, steel, or reinforced-concrete superstructure. It has, however, one very serious limitation, namely, length. On account of its weight and small cross-section, it is difficult, and after certain limits impossible, to move and raise it from a horizontal to a vertical position, and so has only been used in shallow mud and water as in bulkhead construction. To remedy this defect, a type of substructure construction has been devised with all the good features of the full-length pre-cast pile, without its limitations, and can be built at much less cost. This has been patented and is called the socket-pile.

"The pile, as shown, is of reinforced concrete, molded on the ground in either steel or wooden forms prior to driving. It is fitted on the end with a socket for superimposing upon a wooden-pile foundation or base, as shown. The dimensions of the various sections are shown in the following report on tests.

"By this method is obtained a solid concrete construction, from a short distance below the mud-line upward, or in all sections which are liable to decay, to attack by marine insects, or fire.

"As the wooden pile is practically everlasting below the mud, this gives a cheap and easily handled type of construction, particularly adapted to deep mud localities, where the extreme length of concrete piling of the ordinary type prohibits its use. The cost is much less than that of any other possible form of concrete construction and compares favorably, even in first cost, with temporary creosoted wood-pile construction. When the repair cost of the two types is considered, no comparison can be made. (See previous data.)

"The soundings are first made in the mud

in the usual manner, and the length of wooden pile computed so that its top, when fully driven, will be a foot or two below the mud-line. It is then driven to within a short distance of the water-line, and at a point near where the bottom of the socket will come, a collar is placed around it (burlap has proved very satisfactory). The socket pile is then placed over the wooden one and held up about 6 inches while all the space in the socket not occupied by wood is filled with grout through the small pipe shown, which is then capped and the socket-pile dropped to place, forcing, under great pressure, the extra grout into all voids. The combined pile is then driven to place. That this joint, after setting, has more strength than any other section of the pile is proved in the tests. On account of its short length, it is easily and cheaply handled.

"Any type of superstructure can be used, and even for wooden piers this method of construction is cheaper and more efficient than concrete-protected wooden piles."

This type of piling is the invention of K. D. MacLean, formerly Superintendent of Construction and Repair, Board of State Harbor Commissioners, San Francisco.

The piers constructed at the Puget Sound Navy Yard built of concrete have consisted generally of concrete cylinders filled with concrete after sinking, braced together with galvanized rods supporting reinforced beams and floor slabs, and having timber fender piles. The latest one to be constructed, 480 feet in length, with a 210-foot creosoted-pile approach from the shore, is shown in detail in Figs. 380, 381 and 382. The pier, as indicated in Fig. 380, is 80 feet in width and has the supporting cylinders in bents 30 feet center to center, and the cylinders 20 feet center to center transversely, there being four to each bent with a cantilever extension of the main reinforced concrete girder of 10 feet on each side and at the outer end. The cylinders resting upon thirteen piles cut off as shown, above the dredged bottom, are 4 feet in diameter (Fig. 381), with a bell-shaped base covering the piles, which is 11 feet in diameter. The specifications called for these cylinders to be cast on shore, and each having set for thirty days, to be picked up with a floating crane and set in place. After being cleaned out, they were to be filled with concrete of the proportions of about 1–3–5. The main girder, 10 feet in depth and fully reinforced, presented a harder problem in its construction. To avoid the necessity of driving false-work, the author planned the trussed reinforcing, Fig. 382, to support the forms and the weight of the concrete during construction, and before the concrete would become fully self-supporting. The cylinders were to be constructed

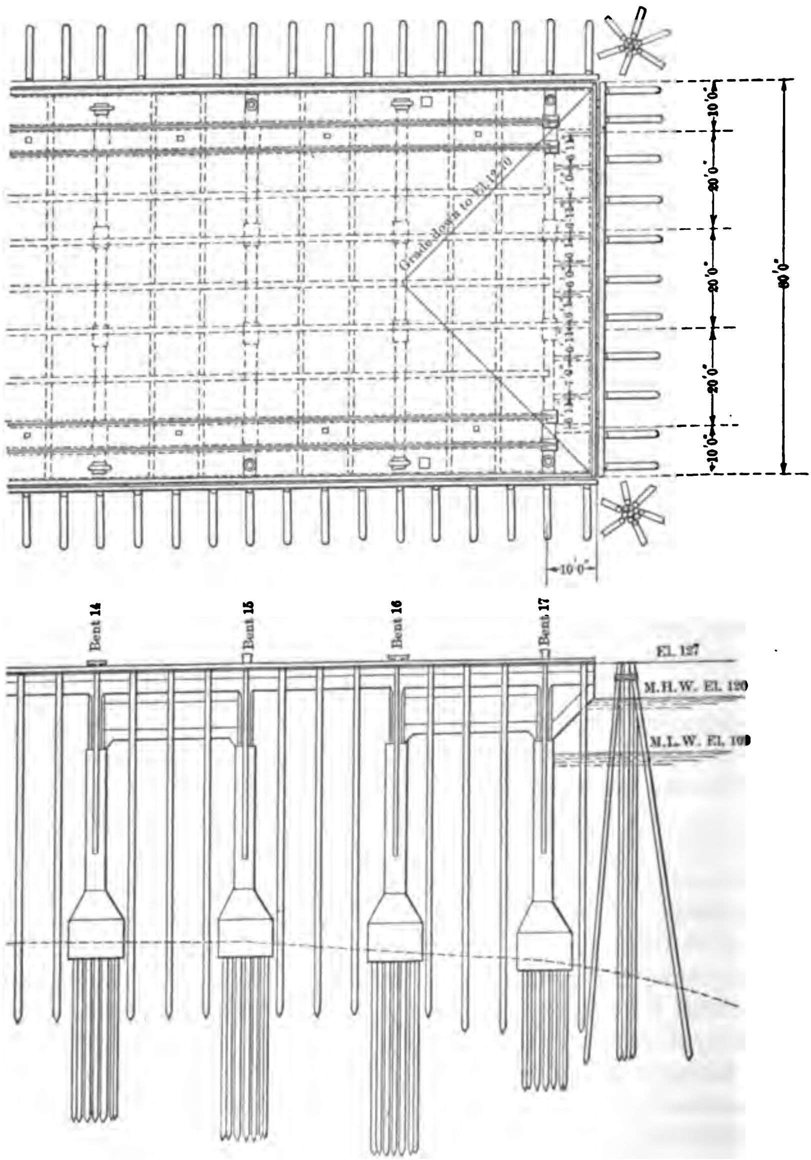


FIG. 380.—NAVY CONCRETE PIER, GENERAL PLAN.

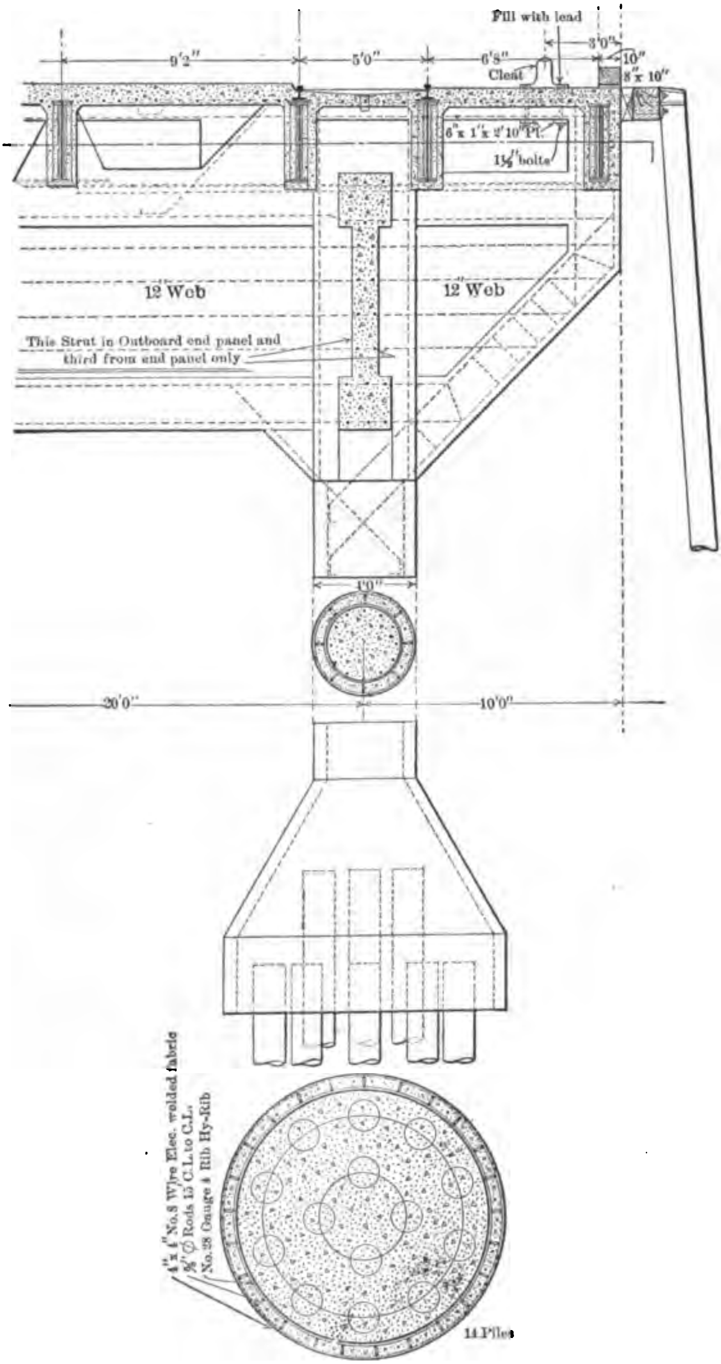


FIG. 381.—NAVY CONCRETE PIER, DETAILS.

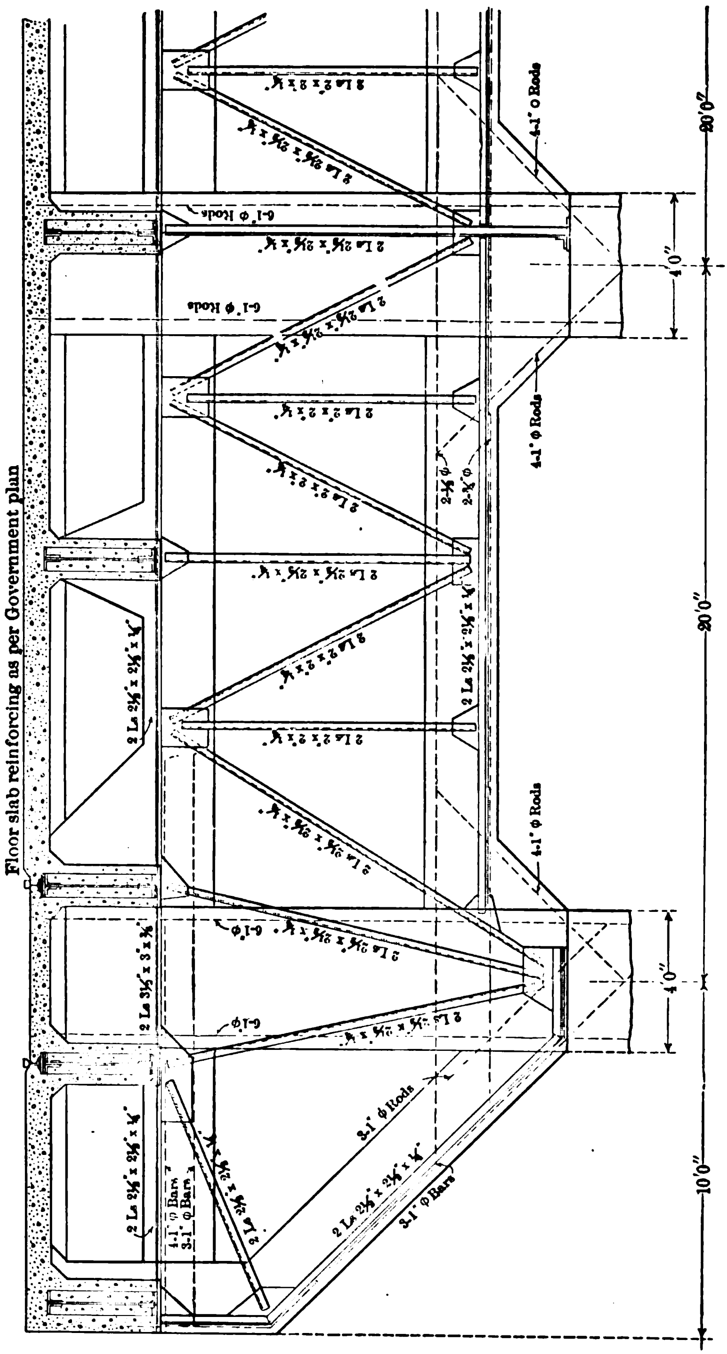


Fig. 382.—Navy Concrete Pier, Proposed Trussed Reinforcing.

6 inches thick; the flanges of the main girder 24 inches thick, the webs 12 inches thick, and the steel stringers buried in concrete about 14 inches thick. The floor slab, $8\frac{1}{2}$ inches thick, is reinforced with $\frac{5}{8}$-inch round rods 8 inches center to center in both directions. The estimate of cost of this pier is given in Chapter XXXIII, and shows the cost of this type of pier to be practically $4 per square foot. The guard timbers and fender piling are of timber as shown. The following clauses from the specifications are reproduced as being worthy of study in the construction of similar piers:

"*Proportions.* Concrete shall be proportioned according to 'Method B' of Specification 59C2. Reinforced concrete in cylinder shells, beams, joists, floor slabs, and covering for plate girders shall be 'Class B' as described in the above specification. Concrete filling for cylinders shall be 'Class A,' as described in the specification. (Appendix IX.)

"*Proportions of concrete* under Method B, which is to be used only when specially required by the specifications for the work, shall be as follows:

"*Class A.* One part cement (allowing 100 pounds to the cubic foot) to six and one-half parts by volume of broken stone combined with a variable proportion of sand.

"*Class B.* One part cement (allowing 100 pounds to the cubic foot) to four and one-fourth parts by volume of broken stone combined with a variable proportion of sand.

"*Concrete Shells.* The reinforced concrete shells shall be cast in substantial, tight forms, held true to line and surface. Reinforcing bars shall be accurately placed and positively secured so as to hold their position while concrete is being placed. Special care shall be taken to prevent voids in the concrete, and the surface shall be continuous, without breaks or pockets. Any defects of this character found upon the removal of forms shall be eliminated by the use of cement mortar worked in with a wooden float.

"*Placing Shells.* After being allowed to set at least thirty days, concrete shells shall be lowered in position by crane, and base sunk to elevations shown. Where cylinders are not supported by piles, should the bearing at the elevation shown on the plans be considered insufficient by the officer in charge, additional compensation will be allowed for any additional depths required, determined as provided under 'Changes' in the general provisions. After the concrete shell has been set and secured in position, all mud or other loose material inside the shell shall be removed to the level of the bottom of the shell by centrifugal pump or otherwise. The bottom

shall then be sealed by a layer of concrete about 2 feet thick, deposited by means of a *trêmie* or bottom-dumping bucket. After the concrete seal has set, the shell shall be pumped out, the reinforcing rods placed, and then filled with concrete deposited in such a manner as to prevent segregation of its component materials.

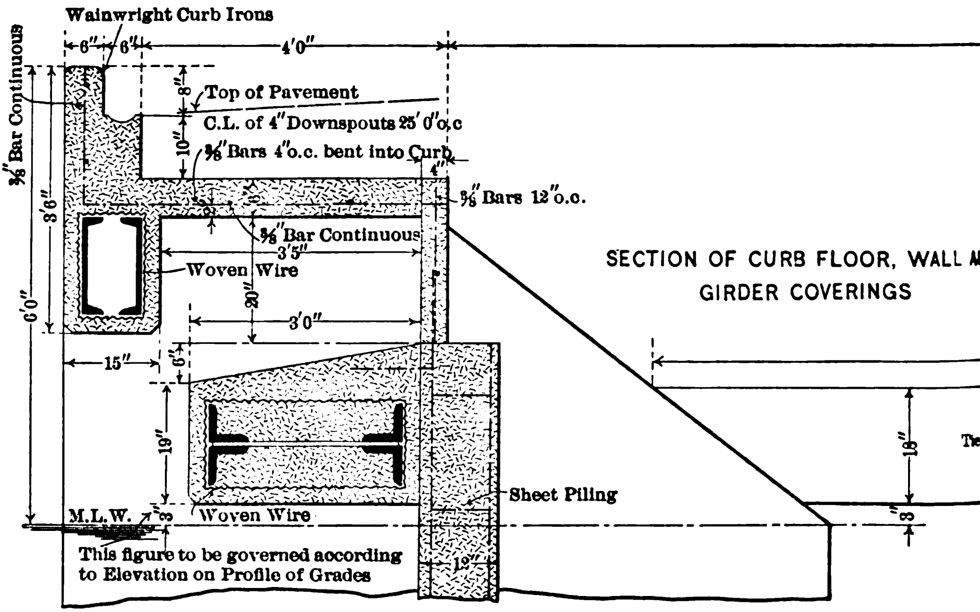
" *Concrete and Steel Superstructure.* The superstructure, consisting of reinforced-concrete caps, steel girders, and reinforced-concrete beams, supporting a reinforced-concrete floor slab, shall be constructed as shown on plans. Concrete for each member below high-water level shall be completely placed in the dry and between two successive high tides. The floor slabs shall be cast in panels the full width

FIG. 383.—BALTIMORE SOLID JETTY PIER.

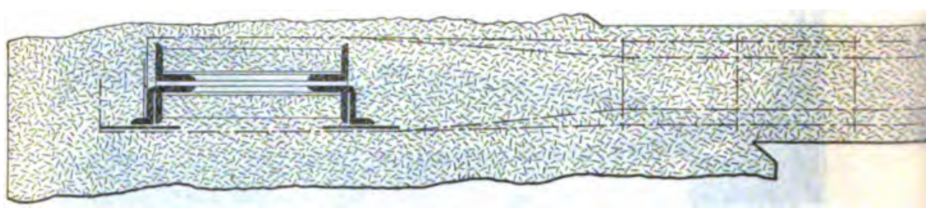
of the pier and in 10-foot lengths. Drain holes shall be left along the rails at intervals of 10 feet. The floor slab shall be finished by rough troweling with a wooden float after having been struck to surface with a straightedge, and all large stones depressed below the surface. The edges of the 10-foot panels shall be neatly rounded with an edging tool.

" *Plate Girders.* Plate girders shall be placed and fastened as shown. There shall be placed between the ends of girders a 1-inch plank of soft wood and the girders then bolted firmly together. Three-eighth-inch round rods shall be fastened around all plate girders at intervals of 15 inches to hold its protective coating of concrete in place."

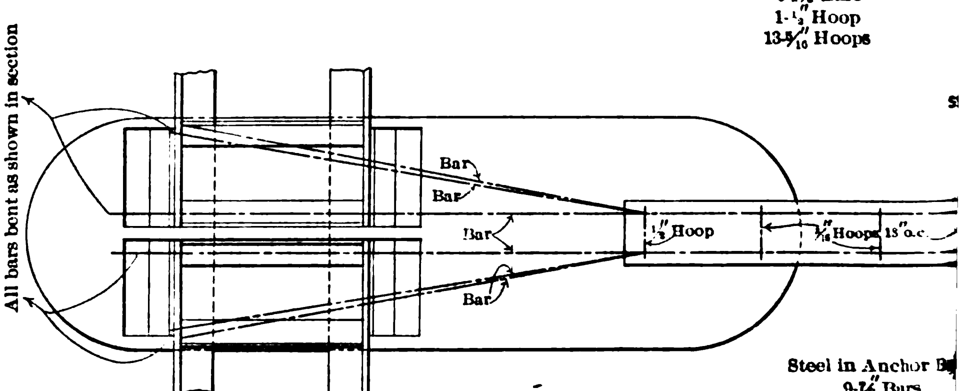
The piers (Fig. 383) constructed by the City of Baltimore



SECTION OF CURB FLOOR, WALL AN[D] GIRDER COVERINGS


SECTION THROUGH TIE SHOWING STEEL

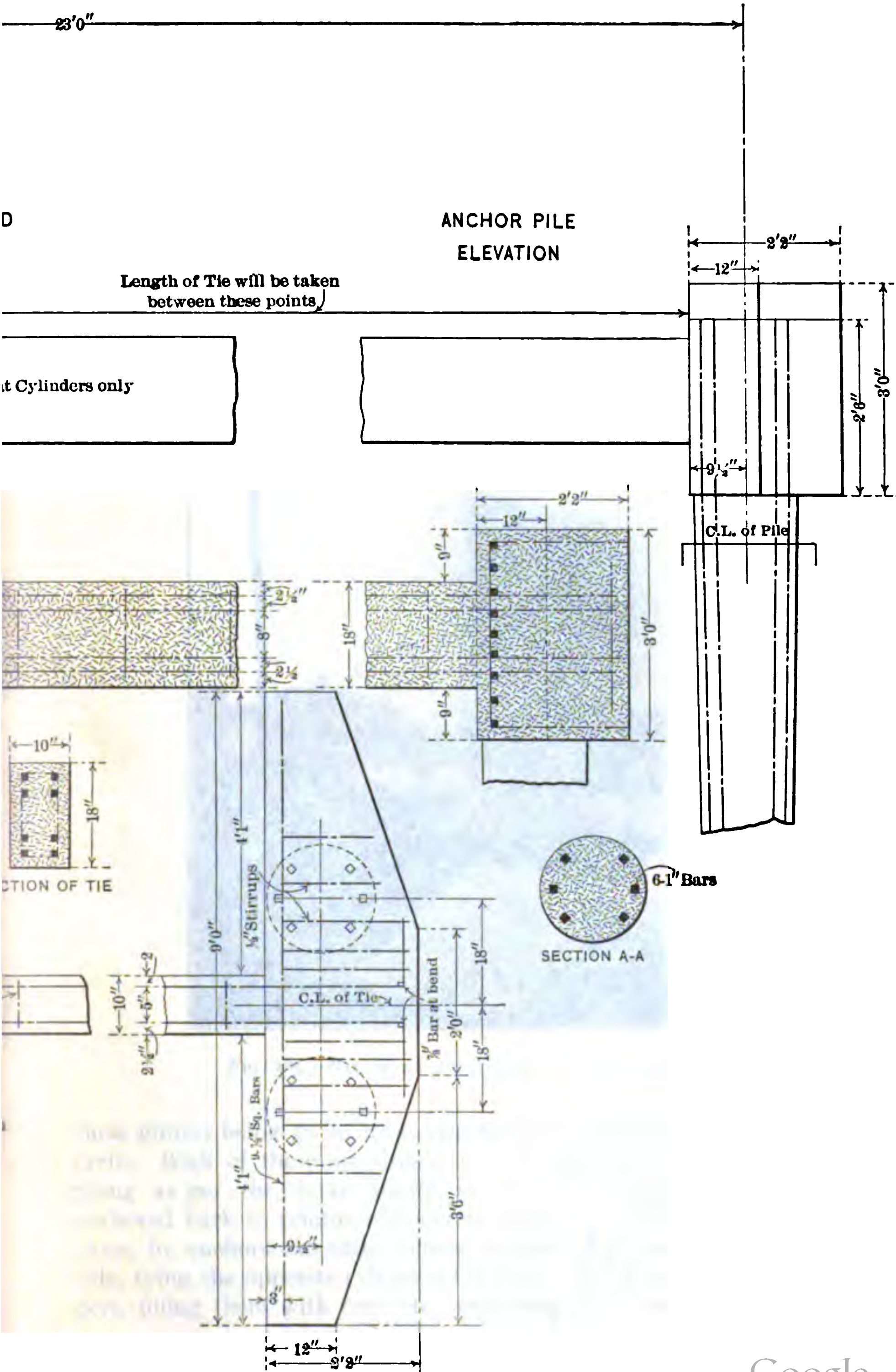
Steel in Tie
8-1⅛″ Bars
1-½″ Hoop
13-5/16″ Hoops



PLAN OF TIE CONSTRUCTION

Steel in Anchor B[lock]
9-⅞″ Bars
10-½″ Stirrups
2-⅞″ Bars

FIG. 384.—BALT[I]



E JETTY PIER DETAILS.

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(To face page 560.)

under Oscar F. Lackey, M. Am. Soc. C.E., Chief Engineer of the Department of Docks, are constructed of the solid jetty type, with the slips dredged to 24 feet alongside. The outside walls of these jetties are constructed by first sinking oblong cylinders (Figs. 384 and 385) 25 feet apart center to center down to the depth shown. (Fig. 386.) They are 3 feet wide by 10 feet long, with rounded ends, and are connected together by horizontal plate girders as shown in section,

FIG. 385.—BALTIMORE JETTY PIER CONSTRUCTION.

these girders being 30 inches in depth, and afterwards buried in concrete. Back of these are driven 12-inch reinforced-concrete sheet-piling as per the detail shown in Fig. 387. The cylinders are anchored back to reinforced-concrete anchors as shown, or in some cases, by anchors extending across the pier (Fig. 388) from side to side, tying the opposite cylinders together. After sinking the cylinders, filling them with concrete, anchoring them back, placing the

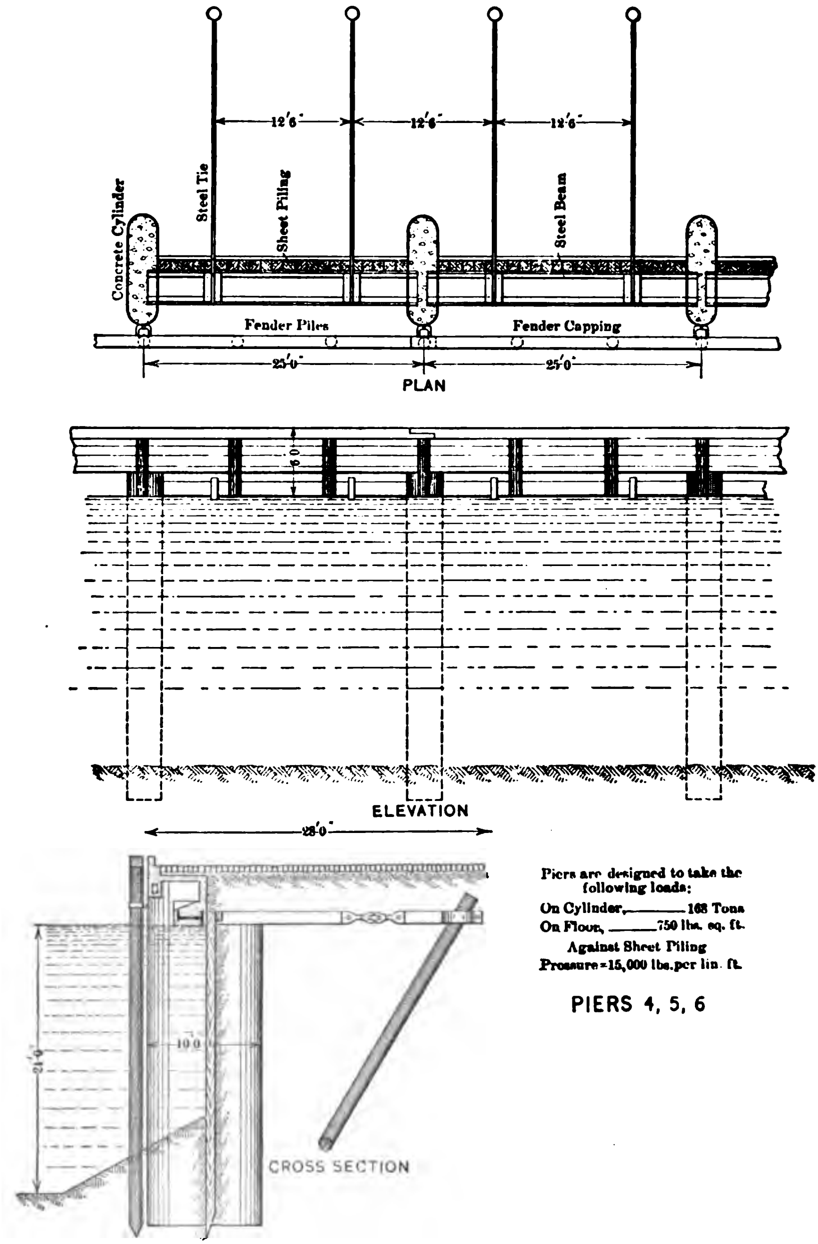
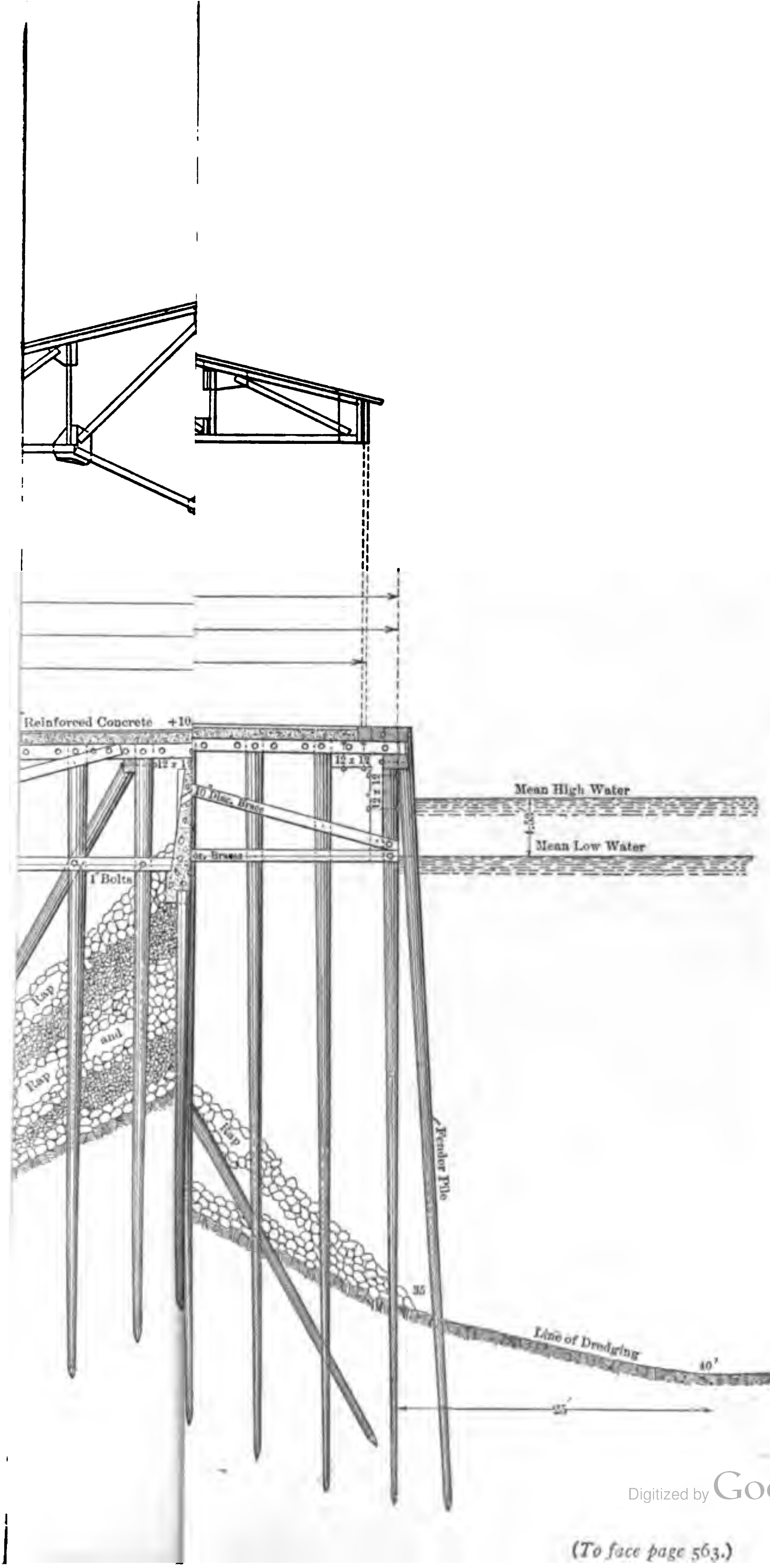


FIG. 386.—BALTIMORE JETTY PIER. CROSS-SECTION.



(To face page 563.)

horizontal girders and the sheet-piling, the space inside the sheet-piling is filled with earth and pavement laid at the elevation shown, and the sheds or warehouses erected as is usual for any structure upon dry land.

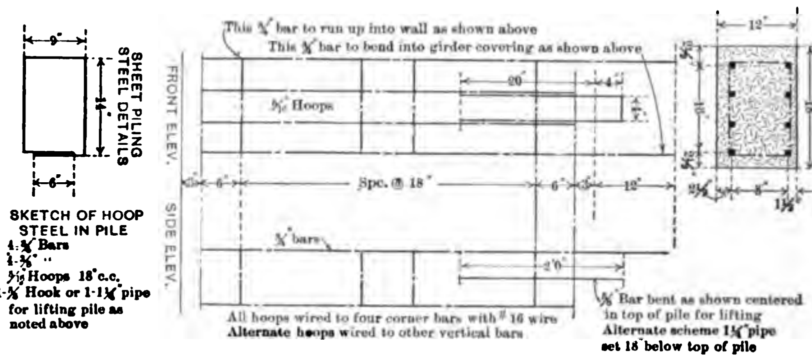


FIG. 387.—BALTIMORE CONCRETE SHEET PILE.

This type of pier is very expensive to construct and is not susceptible to later changes either in the pier itself or in the deepening of the slips alongside to any considerable extent, unless, of course, the construction has been made heavier, and the cylinders and sheet-

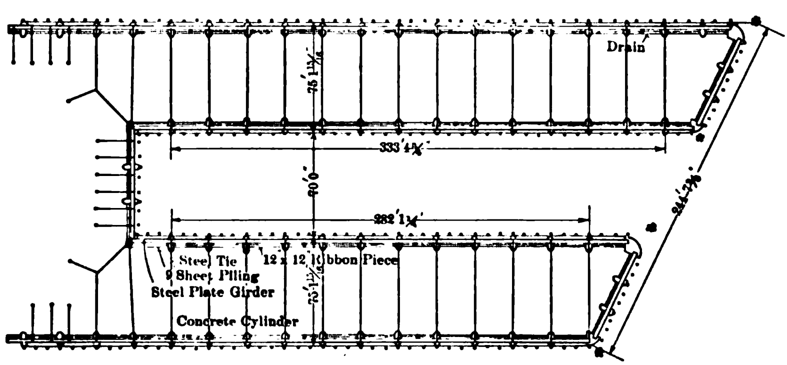


FIG. 388.—BALTIMORE JETTY PIER PLAN.

piling sunk and driven to greater depth to provide for such contingencies.

The type of construction as planned for the South Brooklyn piers in New York Harbor (Fig. 389) is an example of a solid or semi-jetty type, but is such as to lend itself readily to changes in the timber

construction, and in the deepening of the slips should this ever be required, which is not likely, as the depth provided for is 40 feet at low tide.

The concrete seawalls supporting the steel columns of the shed are 73 feet 6 inches center to center, and are supported upon wedge-shaped mounds of riprap rock. Between this riprap and the seawalls, the filling is of selected dredged material on top of which pavement is laid. To prevent spreading, tie-rods are introduced between the seawalls as shown. Previous to the placing of the riprap, the creosoted-piles are driven to support the reinforced-concrete deck under the cantilevers of the shed. Careful study of this type of construction leads to the conclusion that it is much the best of any of the permanent forms that have been described in this chapter. The many details and accessories of pier construction will not be taken up further than what has been illustrated in describing the sub-structures of the various types of piers. The estimated costs of many of the various types of piers described in the foregoing pages will be found in Chapters XXXII and XXXIII.

CHAPTER XXVIII

TIMBER PIERS AND TIMBER PRESERVATION

THE construction of piers of piling and sawed timber is quite common throughout the Western States, and, owing to the cheapness with which they can be built, it is often possible to construct a bridge where, if permanent foundations had to be put in, the expense would be so great as to be prohibitive. Where the piles are driven in the ground outside the water-line they will last from five to seven years, and where they are driven in fresh water they will last for a considerably longer period except that the tops of the piles will very often rot out, have to be cut off, and have blocking substituted. Cedar piles will last much longer and will not rot so quickly at the tops. Where such piers are to be constructed in salt water the piling should be protected from the teredo by some process or other, preferably by creosoting, as that will insure against both rotting and the teredo. It is also advisable in piers constructed on land, or in fresh water, to protect the piles in some way, either by creosoting or by coating them with hot carbolineum avenarius. The abutment pier shown in Figs. 390 and 391 was constructed on the line of the Puget Sound Electric Railway between Seattle and Tacoma, as the end pier to a 200-foot draw span, the bridge and foundations being constructed from plans prepared by the author. The bridge seats come directly over two piles, and these are closely flanked by two additional piles on each side, spaced 2 feet 6 inches from the center line of the trusses each way, while four additional piles are driven in the center of the pier. The piles are capped crosswise with short 12″×12″ caps, and on these are laid the longitudinal 12″×12″ caps which carry the bridge seats. The piles are braced with 4″×12″ diagonal braces boat-spiked to the piles at each intersection. The same method of construction is used for the wings, and the whole face of the pier and wings is planked up with 3″×12″ planking spiked on, with two 8-inch boat-spikes in each plank at each pile. The piling was of Washington fir with the bark peeled off and

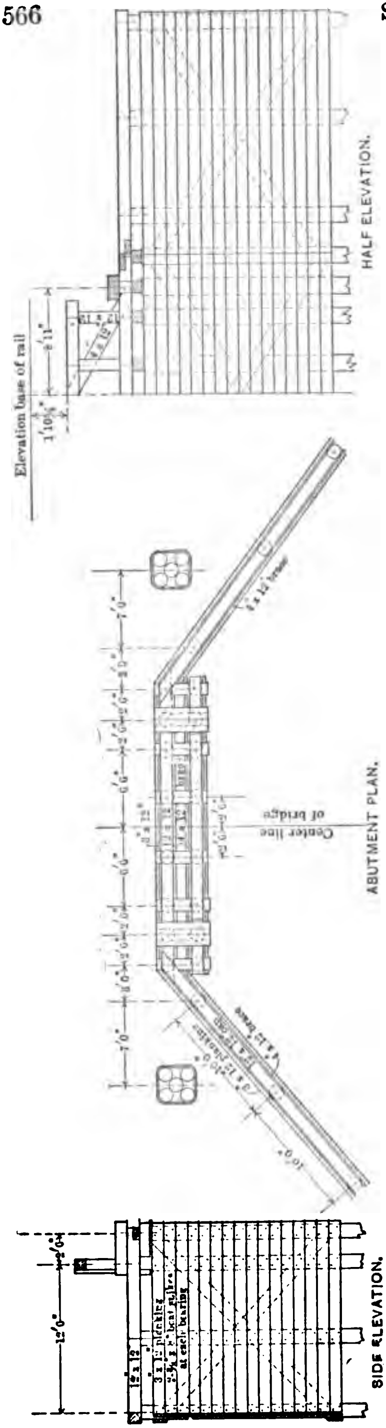


FIG. 390.—PILE PIER, PUGET SOUND ELECTRIC RAILWAY.

treated with hot carbolineum avenarius. These piles were driven to a firm bearing.

The timber was No. 1 merchantable Washington fir, free from sap, wind-shake, pitch seams or other imperfections that might impair its strength and durability. In addition to the planking on the face of the pier, to protect it, five-pile dolphins were driven, as shown, 10 feet from the center of the end piles of the pier proper, and wrapped together with wire cables. Carefully constructed as these piers were, it is believed that their life will be from ten to twelve years at the least calculation. The piers, as well as the bridges, were calculated to carry train-loads consisting of Cooper's E–27 Loading. The cost of the piling was seven cents per lineal foot delivered at the bridge site, to which should be added the cost of coating, while the cost of the timber was $12 delivered at the bridge site. The cost of driving the piles was approximately $4 each for driving and cutting off, and the cost of placing the timber was approximately $4 per thousand feet board measure. Another pier on the same road is shown in Fig. 394.

In the construction of the Raging River bridge and trestle, on the line of the Northern Pacific Railroad, between Seattle and Snoqualmie

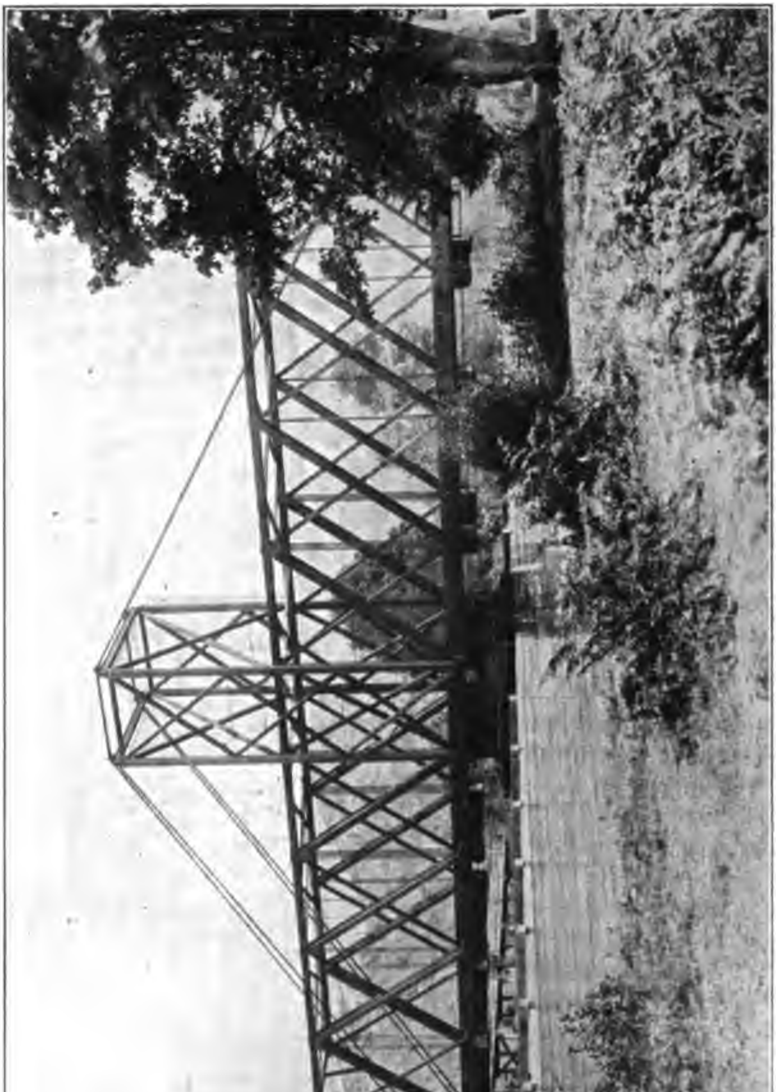


Fig. 391.—Duwamish Draw, Puget Sound Electric Railway.

Falls, it was necessary to cross the stream with a Howe truss span supported on timber towers or piers, a detail of which is shown in Fig. 392. The original trestle and bridge was left standing about thirteen years, or about three years longer than the life of the timber, and fell down during the passage of a log train. The fall was caused primarily by some logs falling off of the train and knocking out some of the batter posts of the trestle, after which the entire trestle col-

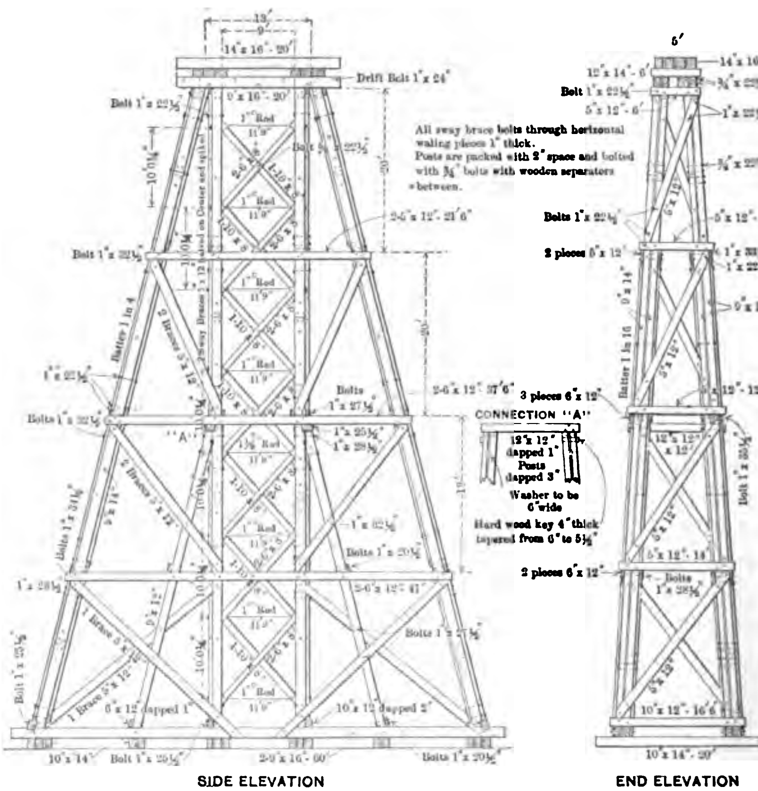


FIG. 392.—RAGING RIVER BRIDGE PIER, SNOQUALMIE FALLS LINE.

lapsed, although it had outlived its usefulness owing to the age of the timber. This occurred previous to the acquisition of the road by the Northern Pacific Railway Company. The trestle (Fig. 393) was reconstructed by the author in approximately thirty days' working time from the starting of the work until the passage of trains, although the actual time of the entire completion of the work covered a period of between six and seven weeks. All of the timber was still standing

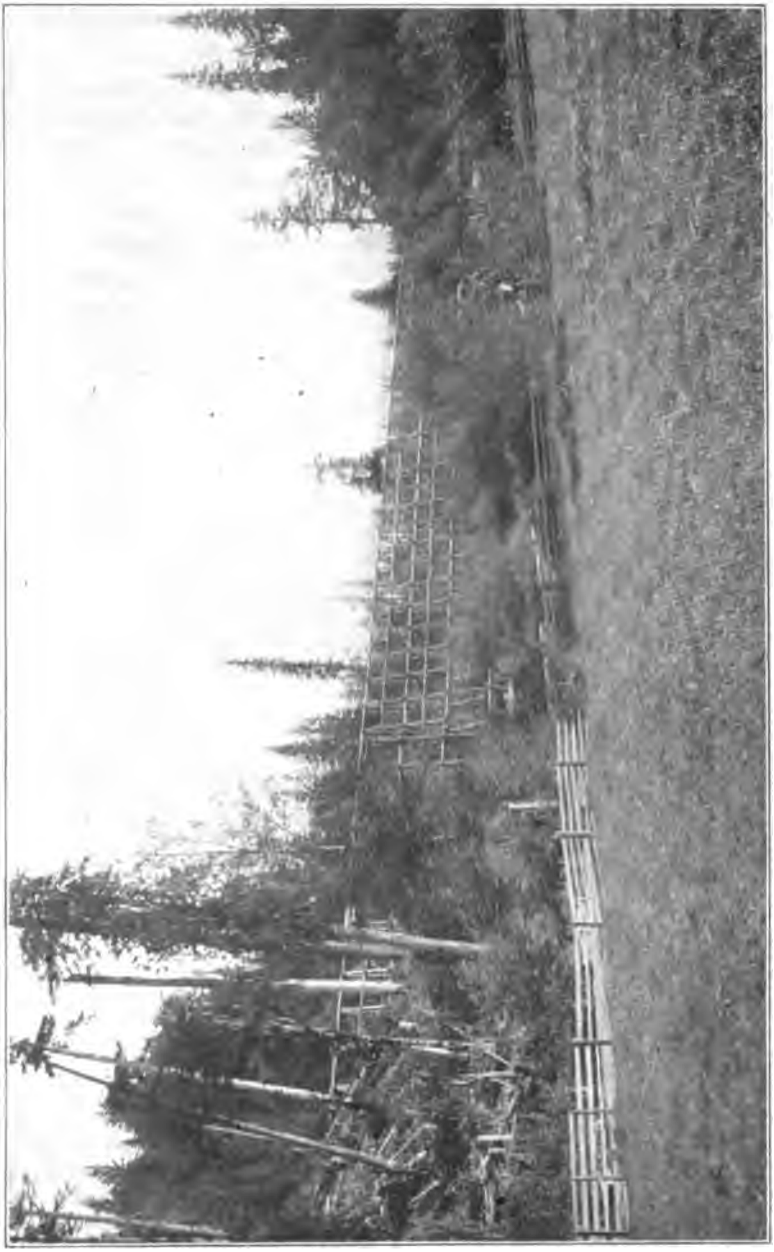


FIG. 393.—RAGING RIVER BRIDGE, NORTHERN PACIFIC RAILWAY.

in the forests when the contract was taken, and right of way was given the log trains carrying this timber over the road, so that it was turned out with great rapidity, and the 800,000 feet B.M. of timber that was used for the trestle and bridge was landed at the bridge site in a remarkably short space of time. The piers were carefully framed before erection was started on them, by laying them out on the ground and framing the work complete down to the boring of the bolt holes. The plumb posts were braced together transversely with Howe truss bracing, consisting of timber diagonals formed of two 6″×8″ sticks in one direction and one stick 8″×10″ in the other

FIG. 394.—PIERS OF GEORGETOWN BRIDGE.

direction. The batter posts were braced to the plumb posts with 5″×12″ and 6″×12″ girts, and 5″×12″ diagonal braces. The plumb posts and batter posts were each formed of two 9″×14″ sticks packed together. Additional batter posts 9″×12″ ran to longitudinal girts at half the height of the pier. The truss-rods of the Howe truss bracing varied from 1 inch to $1\frac{1}{8}$ inches in size, while most of the bolts used for fastening the work together were 1 inch. The tower rested upon 10″×14″ mud-sills and 9″×16″ main-sills with caps of the same size. The ends of all timbers were white-leaded, but none of it was painted or treated in any way. Such a pier as this should be good for from eight to ten years' ser-

vice for ordinary railroad traffic, although new work of this character, on even the Western railroads, at the present time is easily constructed of steel.

The designing of timber trestles, which would cover piers of the character under discussion, is most fully treated in Bulletin No. 12 of the U. S. Department of Agriculture, Timber Physics Series, the title of the Bulletin being "Economical Designing of Timber Trestle Bridges." The treatise is the work of A. L. Johnson, C.E.

The strength of the bridge and trestle timbers has been fully treated in a report by a committee of the American International Association of Railway Superintendents of Bridges and Buildings on the "Strength of Bridge and Trestle Timbers:"

"The test data at hand and the summary of criticisms of leading authorities seem to indicate the general correctness of the following conclusions:

"(1) Of all structural materials used for bridges and trestles, timber is the most variable as to the properties and strength of the different pieces classed as belonging to the same species; hence it is impossible to establish close and reliable limits for each species.

"(2) The various names applied to one and the same species in different parts of the country lead to great confusion in classifying or applying results of tests.

"(3) Variations in strength are generally directly proportional to the density or weight of timber.

"(4) As a rule, a reduction of moisture is accompanied by an increase in strength; in other words, seasoned lumber is stronger than green lumber.

"(5) Structures should be, in general, designed for the strength of green or moderately seasoned lumber of average quality and not for a high grade of well-seasoned material.

"(6) Age and use do not destroy the strength of timber unless decay or season checking takes place.

"(7) Timber, unlike materials of a more homogeneous nature, as iron and steel, has no well-defined limit of elasticity. As a rule, it can be strained very near to the breaking point without serious injury, which accounts for the continuous use of many timber structures with the material strained far beyond the usually accepted safe limits. On the other hand, sudden and frequently inexplicable failures of individual sticks at very low limits are liable to occur.

"(8) Knots, even when sound and tight, are one of the most objectionable features of timber, both for beams and struts. The full-size tests of every experimenter have demonstrated not only

that beams break at knots, but that invariably timber struts will fail at a knot or owing to the proximity of a knot, by reducing the effective area of the stick and causing curly and cross-grained fibers, thus exploding the old practical view that sound and tight knots are not detrimental to timber in compression.

"(9) Excepting in top logs of a tree or very small and young timber, the heart-wood is, as a rule, not as strong as the material farther away from the heart. This becomes more generally apparent, in practice, in large sticks with considerable heartwood cut from old trees in which the heart has begun to decay or been wind-shaken. Beams cut from such material frequently season check along middle of beam and fail by longitudinal shearing.

"(10) Top logs are not as strong as butt logs, provided the latter have sound timber.

"(11) The results of compression tests are more uniform and vary less for one species of timber than any other kind of test; hence, if only one kind of test can be made, it would seem that a compressive test will furnish the most reliable comparative results.

"(12) Long timber columns generally fail by lateral deflection or 'buckling' when the length exceeds the least cross-sectional dimension of the stick by 20; in other words, when the column is longer than 20 diameters. In practice the unit stress for all columns over 15 diameters should be reduced in accordance with the various rules and formulæ established for long columns.

"(13) Uneven end bearings and eccentric loading of columns produce more serious disturbances than are usually assumed.

"(14) The tests of full-size long compound columns, composed of several sticks bolted and fastened together at intervals, show essentially the same ultimate unit resistance for the compound column as each component stick would have if considered as a column by itself.

"(15) More attention should be given in practice to the proper proportioning of bearing areas; other in words, the compressive bearing resistance of timber with and across grain, especially the latter, owing to the tendency of an excessive crushing stress across grain to indent the timber, thereby destroying the fiber and increasing the liability to speedy decay, especially when exposed to the weather and the continual working produced by moving loads.

"The aim of your committee has been to examine the conflicting test data at hand, attributing the proper degree of importance to the various results and recommendations, and then to establish a set of units that can be accepted as fair average values, as far as

known to-day, for the ordinary quality of each species of timber and corresponding to the usual conditions and sizes of timbers encountered in practice. The difficulties of executing such a task successfully cannot be overrated, owing to the meagerness and frequently the indefiniteness of the available test data, and especially the great range of physical properties in different sticks of the same general species, not only due to the locality where it is grown, but also to the condition of the timber as regards the percentage of moisture, degree of seasoning, physical characteristics, grain, texture, proportion of hard and soft fibers, presence of knots, etc., all of which affect the question of strength.

" Your committee recommends, upon the basis of the test data at hand at the present time, the average units for the ultimate breaking stresses of the principal timbers used in bridge and trestle constructions shown in the accompanying table.

" In addition to the units given in the table, attention should be called to the latest formulæ for long timber columns, mentioned more particularly in the appendix to this report, which formulæ are based upon the results of the more recent full-size timber-column tests, and hence should be considered more valuable than the older formulæ derived from a limited number of small-size tests. These new formulæ are Professor Burr's, Appendix I; Professor Ely's, Appendix J; Professor Stanwood's, Appendix K; and A. L. Johnson's, Appendix V; while C. Shaler Smith's formulæ will be better understood after examining the explanatory notes contained in Appendix L. (The formula recommended for use by the author is nearly that of Professor Burr, or $p=6000-70\frac{l}{d}$, in which $p=$ ultimate strength per square inch, $l=$length of column in inches, and $d=$the least dimension of the column in inches. This is for Washington fir or similar timber, between the limits of $20\frac{l}{d}$ and $60\frac{l}{d}$.)

" Attention should be called to the necessity of examining the resistance of a beam to longitudinal shearing along the neutral axis, as beams under transverse loading frequently fail by longitudinal shearing in the place of transverse rupture.

" In addition to the ultimate breaking unit stress the designer of a timber structure has to establish the safe allowable unit stress for the species of timber to be used. This will vary for each particular class of structures and individual conditions. The selection of the proper ' factor of safety ' is largely a question of personal judgment

and experience, and offers the best opportunity for the display of analytical and practical ability on the part of the designer. It is difficult to give specific rules. The following are some of the controlling questions to be considered:

"The class of structure, whether temporary or permanent, and the nature of the loading, whether dead or live. If live, then whether the application of the load is accompanied by severe dynamic shocks and pounding of the structure; whether the assumed loading for calculation is the absolute maximum, rarely to be applied in practice, or a possibility that may frequently take place. Prolonged heavy, steady loading, and also alternate tensile and compressive stresses in the same place, will call for lower averages. Information as to whether the assumed breaking stresses are based on full-size or small-size tests or only on interpolated values, averaged from tests of similar species of timber, is valuable in order to attribute the proper degree of importance to recommended average values. The class of timber to be used and its condition and quality. Finally, the particular kind of strain the stick is to be subjected to and its position in the structure with regard to its importance and the possible damage that might be caused by its failure.

"In order to present something definite on this subject, your committee presents the accompanying table, showing the average safe allowable working unit stresses for the principal bridge and trestle timbers, prepared to meet the average conditions existing in railroad timber structures, the units being based upon the ultimate breaking unit stresses recommended by your committee and the following factors of safety, viz.:

Tension with and across grain	10
Compression with grain	5
Compression across grain	4
Transverse rupture, extreme fiber stress	6
Transverse rupture, modulus of elasticity	2
Shearing with and across grain	4

"In conclusion, your committee desires to emphasize the importance and great value to the railroad companies of the country of the experimental work on the strength of American timbers being conducted by the Forestry Division of the United States Department of Agriculture, and to suggest that the American Association of Railway Superintendents of Bridges and Buildings indorse this view by official action and lend its aid in every way possible to encourage the vigorous continuance of this series of

Government tests, which bids fair to become the most reliable and useful work on the subject of strength of American timbers ever undertaken. With additional and reliable information on this subject far-reaching economies in the designing of timber structures can be introduced, resulting not only in a great pecuniary saving to the railroad companies, but also offering a partial check to the enormous

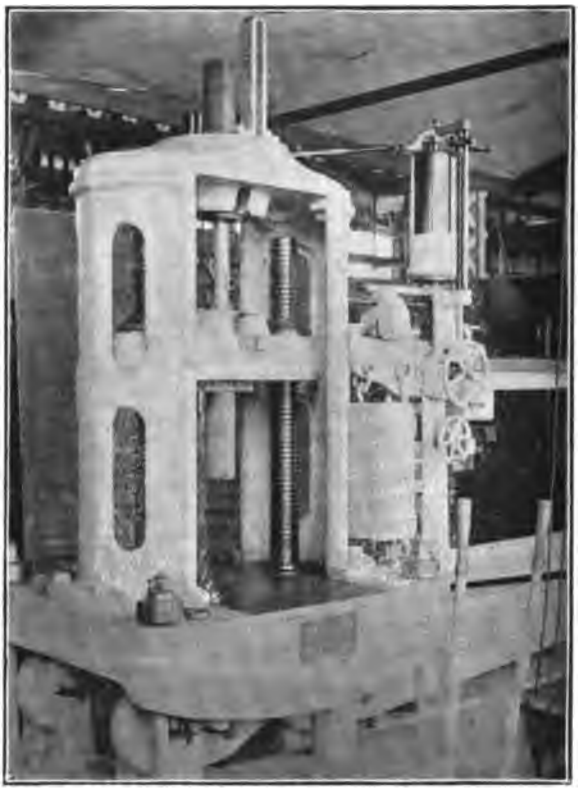


FIG. 395.—TENSILE TEST, DOUGLAS FIR.

consumption of timber and the gradual diminution of our structural timber supply."

A very complete series of tests were made by Frank W. Hibbs, Naval Constructor, U. S. N., at the Puget Sound Navy Yard, on the "Comparative Tests of Yellow Pine and Puget Sound Fir." This is fully published in a paper of the Pacific Northwest Society of Engineers, to which the reader is referred, as it is impossible to quote even results here. Fig. 395 shows the method used for making

the tensile tests, and Fig. 396 the method used for making the transverse tests. The conclusions drawn from these tests were as follows:

1st. Strength. Douglas fir is generally equal to yellow pine, and superior to it in some essential particulars.

2d. Elasticity. Douglas fir is decidedly more elastic than yellow pine.

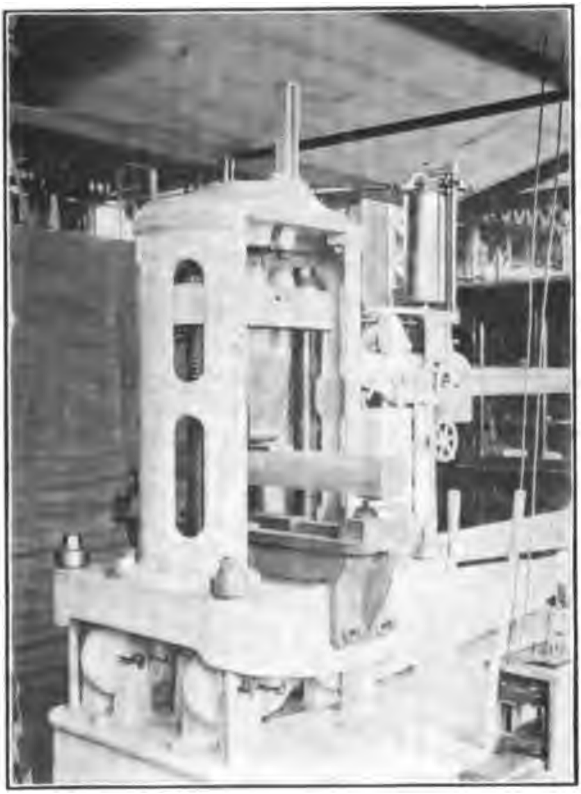


FIG. 396.—TRANSVERSE TEST, DOUGLAS FIR.

3d. Toughness. Douglas fir is far superior to yellow pine as regards toughness.

4th. Wearing qualities. Yellow pine is superior to Douglas fir, especially when moisture is present.

5th. Lasting qualities. Yellow pine is superior to Douglas fir, on account of the greater amount of pitch that it contains.

6th. Weight. Douglas fir is 14 per cent. lighter than yellow pine.

TABLE LXIV.—AVERAGE ULTIMATE BREAKING UNIT STRESSES IN POUNDS PER SQUARE INCH RECOMMENDED BY THE COMMITTEE ON "STRENGTH OF BRIDGE TRESTLE TIMBERS," AMERICAN ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS, FIFTH ANNUAL CONVENTION, NEW ORLEANS, OCTOBER, 1895.

Kind of Timber.	Tension.		Compression.			Tranvserse Rupture.		Shearing.	
			With Grain.		Across Grain.				
	With Grain.	Across Grain.	End Bearing.	Columns under 15 Diameters.		Extreme Fiber Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
White oak	10,000	2,000	7,000	4,500	2,000	6,000	1,100,000	800	4,000
White pine	7,000	500	5,500	3,500	800	4,000	1,100,000	400	2,000
Southern, longleaf, or Georgia yellow pine	12,000	600	8,000	5,000	1,400	7,000	1,700,000	600	5,000
Douglas, Oregon, and Washington fir or pine:									
Yellow fir	12,000	.....	8,000	6,000	1,200	6,500	1,400,000	600	
Red fir	10,000	.....	.....	.....	.....	5,000			
Northern or shortleaf yellow pine	9,000	500	6,000	4,000	1,000	6,000	1,200,000	400	4,000
Red pine	9,000	500	6,000	4,000	800	5,000	1,200,000		
Norway pine	8,000	.....	6,000	4,000	800	4,000	1,200,000		
Canadian (Ottawa) white pine	10,000	.....	.....	5,000	.....	.....	....	350	
Canadian (Ontario) red pine	10,000	.....	.....	5,000	.....	5,000	1,400,000	400	
Spruce and Eastern fir	8,000	500	6,000	4,000	700	4,000	1,200,000	400	3,000
Hemlock	6,000	.....	.....	4,000	600	3,500	900,000	350	2,500
Cypress	6,000	.....	6,000	4,000	700	5,000	900,000		
Cedar	8,000	.....	6,000	4,000	700	5,000	700,000	.....	1,500
Chestnut	9,000	.....	.....	5,000	900	5,000	1,000,000	600	1,500
California redwood	7,000	.....	.....	4,000	800	4,500	700,000	400	
California spruce	.....	.....	.....	4,000	.....	5,000	1,200,000		

TABLE LXV.—AVERAGE SAFE ALLOWABLE WORKING UNIT STRESSES IN POUNDS PER SQUARE INCH RECOMMENDED BY THE COMMITTEE ON "STRENGTH OF BRIDGE AND TRESTLE TIMBERS," AMERICAN ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS, FIFTH ANNUAL CONVENTION, NEW ORLEANS, OCTOBER, 1895.

Kind of Timber.	Tension.		Compression.			Transverse Rupture.		Shearing.	
			With Grain.						
	With Grain.	Across Grain.	End Bearing.	Columns under 15 Diameters.	Across Grain.	Extreme Fiber Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
Factor of safety	10	10	5	5	4	6	2	4	4
White oak	1,000	200	1,400	900	500	1,000	550,000	200	1,000
White pine	700	50	1,100	700	200	700	500,000	100	500
Southern, longleaf, or Georgia yellow pine	1,200	60	1,600	1,000	350	1,200	850,000	150	1,250
Douglas, Oregon, and Washington fir or pine:									
Yellow fir	1,200	.....	1,600	1,200	300	1,100	700,000	150	
Red fir	1,000	.....	.....	.....	.....	800			
Northern or Shortleaf yellow pine	900	50	1,200	800	250	1,000	600,000	100	1,000
Red pine	900	50	1,200	800	200	800	600,000		
Norway pine	800	.....	1,200	800	200	700	600,000		
Canadian (Ottawa) white pine	1,000	.....	.....	1,000	.....	.....	....	100	
Canadian (Ontario) red pine	1,000	.....	.....	1,000	.....	800	700,000	100	
Spruce and Eastern fir	800	50	1,200	800	200	700	600,000	100	750
Hemlock	600	.....	.....	800	150	600	450,000	100	600
Cypress	600	.....	1,200	800	200	800	450,000		
Cedar	800	.....	1,200	800	200	800	350,000	.....	400
Chestnut	900	.....	.....	1,000	250	800	500,000	150	400
California redwood	700	.....	.....	800	200	750	350,000	100	
California spruce	......	.....	.....	800	.....	800	600,000		

The main reason why timber is not used in many cases is on account of its short life. Upon exposure to the elements it decays quite rapidly, lasting on the average about ten years in structures, and when used for railroad ties only about seven years. If the surface is protected from the rain by painting, this closes up the pores and causes heart rot from the moisture that is retained inside the stick of timber. For this reason paint should seldom be used upon a

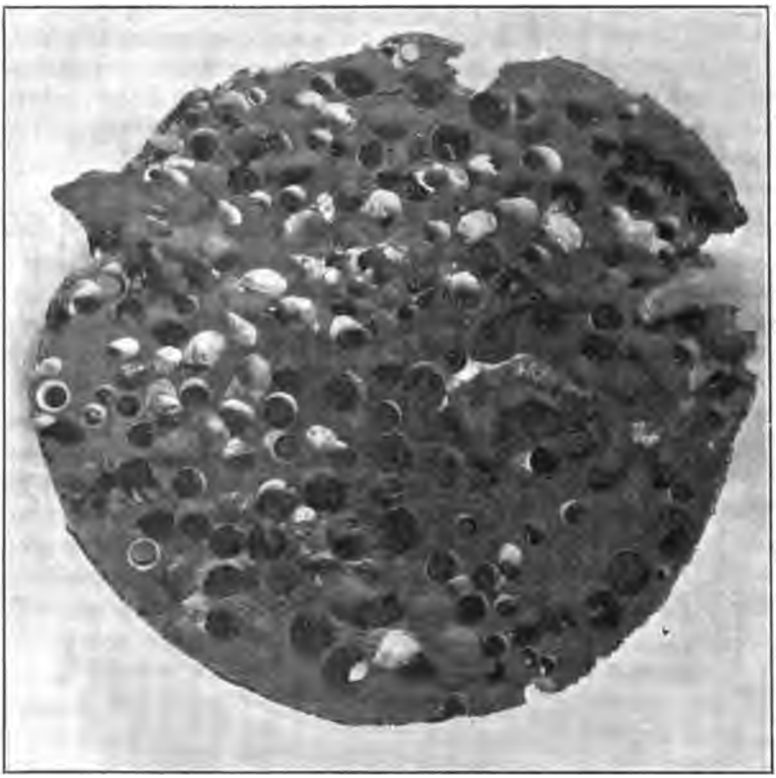


FIG. 397.—SECTION OF TEREDO-EATEN TIGHT BARK PILE.

timber structure of any kind. Where timber is placed in salt water it is destroyed from other and quite different causes. The marine animals which are most destructive are the limnora, which eats off the surface of the timber near the water line, and the teredo, which eats out the interior. Untreated peeled piling driven in salt water become so badly eaten by the teredo in two or three months as to break off under almost no load at all, if this load be applied transversely. The view, Fig. 397, is of a section of piling which was eaten

off in less than two years, it having been driven with the bark on. The bark is a good protection, and some of the methods of protecting piling against the teredo are based upon the plan of providing piles with an artificial bark formed of burlap, treated with asphaltum and wrapped about the piles with wires. The best protection, however, is creosoting, or the impregnating of piling or timber with creosote or the dead oil of coal tar.

The amount of creosote used varies from 10 to 20 pounds per cubic foot, from 12 to 16 pounds being the ordinary amount specified. For the preservation of timber simply against rot, 10 or 12 pounds is sufficient, while many engineers think it necessary to use from 16 to 20 pounds for protection against the teredo. Soft timber takes creosote readily, but some of the harder kinds, or those with hard fibers, will not take up the creosote so easily, so that it would seem advisable to specify the amount of penetration the creosote should have from the surface of the wood inwards instead of specifying the amount per cubic foot.

A penetration of five-eighths of an inch forms a satisfactory protection against the teredo.

As the other methods of preserving timber are less used, they will only be mentioned, and in case the engineer should find a plant available for preserving timber by one of the other methods its processes can readily be compared with those of creosoting. These methods are Kyanizing, or bichloride of mercury process; Burnettizing, or zinc chloride process; and Margaryizing, or sulphate of copper process. None of these, however, is so good as creosoting, inasmuch as the preserving material dissolves out of the timber and leaves it to decay. This feature is very strongly brought out in some experiments that have been made with Egyptian mummies, as when all of the embalming material has been extracted, the mummies at once decay rapidly.

Creosoting plants are found in almost every section of the country, and a description of the methods followed at any one of these works practically covers the methods employed at others.

The description given by the Norfolk Creosoting Company is quoted here in full:

"The preservation of timber by the dead oil of coal-tar process, as carried on by all well-equipped creosoting plants, consists of two distinct operations—the preparation of the wood, and its impregnation with the preservative. The preparation of the wood necessary for the proper reception of the preserving substance is the removal of all those portions of the tissue which are subject to fermentative

action. This consists of the extraction of the liquids and semi-liquids occupying the interfibrous spaces, and constituting the very immature portions of the wood, without softening the cement binding of the fibrillæ, or bundles of cellulose tissue, forming the solid or fully matured part. Upon the successful accomplishment of this entirely depends the value of artificially preserved wood for structural purposes. If this step of the operation is conducted at too low a temperature, or for too short a time, the sap or liquid part nearest the surface will only be extracted, the consequence of which will be an insufficient space for receiving the preservative. If, on the other hand, the operation is carried on at too high a temperature, or for too long a time, the resinous portion of the bundles of fibrillæ will be softened and the wood lose its elasticity in just the proportion that the coherence of the fibrillæ is lessened. The temperature should never be less than 100° C. nor exceed 130° C. Of the two possible methods for the removal of the undesirable portions of the timber—exposure to currents of dry air, and steaming under pressure with an after-drying in a vacuum—the latter is now the universal practice. While the first-named plan may seem the more rational, and the one least likely to modify injuriously the physical structure, such is not the case. Under proper manipulation, a more thorough desiccation, without harmful change of the organic structure, can be accomplished in twelve hours less by the latter process, than is ever possible with air drying, which, under the most favorable circumstances, is a long-drawn-out operation, and cannot do more than extract the water from that portion of the sap which has not yet reached the semi-solid stage, thus leaving in the tissues of the wood a very considerable amount of resinous matter which occupies space that should be ready to receive the creosote-oil. The consequence of this is a failure of the oil to reach many of the interfibrous passages, which are either left empty or are filled with the gelatinous part of the half-matured growth cells in which are to be found the conditions that make putrefaction possible. In order to remove the sap from wood, it is first necessary to vaporize it and then to bring about such external circumstances which shall allow outflow of all gaseous matter from the interior of the wood. In order to vaporize the sap it is necessary to break down the walls of the cells containing the liquid and semi-liquid substances. This is readily accomplished through the agency of heat applied through the medium of a moist steam-bath, at such a pressure as to keep the temperature of the wood, and its surrounding atmosphere, somewhat above the boiling-point of the sap. The maintenance of this condition for a few hours is found to be quite

sufficient to break down the sap-cell tissue and to vaporize all those constituents that it is desirable to withdraw. This point having been reached, the steam-bath is discontinued; and the temperature being maintained at, or slightly above, the vaporizing-point of the sap, the pressure of the atmosphere surrounding the wood within the chamber is reduced below that of the interior of the wood. The result of this condition is an outflow of vapor and air, continuing until equilibrium is restored. This equilibrium is prevented by the use of an exhaust pump until the absence of aqueous vapor in the discharge from the pump indicates the completion of the operation. At this stage the wood-tissue is in a state very like that of a sponge cleared of hot water; every pore is gaping open and ready to receive the oil.

"In the practice of the Norfolk Creosoting Company the most carefully dried lumber is steamed and subjected to the action of the heated 'vacuum' in order that there may be had that thorough and uniform penetration of the preserving liquid that is essential to the highest efficiency of the product. The timber having been thus prepared the creosote-oil is admitted to the chamber, which is still kept under the influence of the vacuum pump, at a temperature somewhat above the boiling-point of the sap, at the pressure then existing in the chamber. As the hot oil envelopes the wood and enters the interfibrous spaces, the aqueous vapor yet remaining in the wood, by reason of its less specific gravity, rises to the top of the containing chamber and is withdrawn by the pump. By the time that the chamber is entirely filled with oil, all the remaining moisture has escaped. The exhaust pump is stopped and, in order to facilitate the absorption of the oil by the wood, a pressure pump is set to work supplying oil to the chamber at such pressure as may be desired. This operation is continued until the requisite amount of oil has been put into the timber. The chamber is then opened and the timber withdrawn. The apparatus is then ready for further use.

"The successful conduct of the operation above outlined exacts the most careful attention and skillful management, supplemented by adequate and suitable applicances. The wide divergence in the characteristics of timber; the varying amounts of sap, due to the lapse of time since, and the season in which the tree was felled; its possible subsequent immersion in water for a longer or shorter time; the character of the soil and the conditions under which the tree grew, whether in a dense forest or a comparatively open country, whether it is of a rapid, even growth, or a slow intermittent one, are all factors contibuting to a more or less perfect product. To the

experienced operator these conditions indicate, in each case, the proper course to be pursued. Failure to observe and to take them into consideration is to invite indifferent, uncertain and in the end unsatisfactory results. Of equal importance is a proper understanding

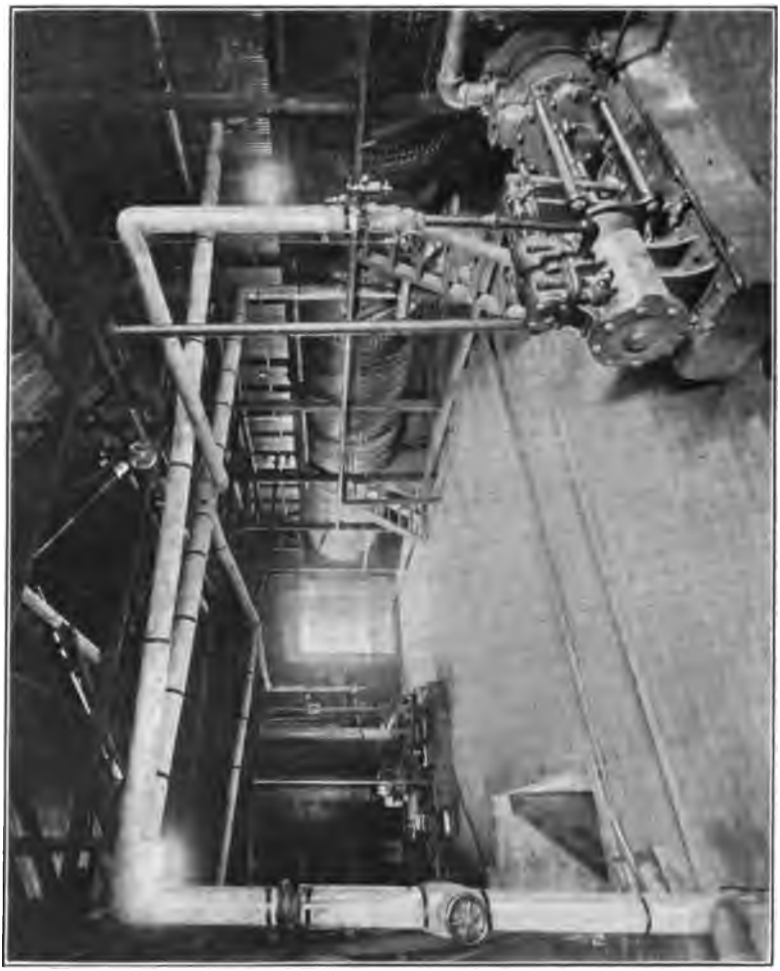


FIG. 398.—PLANT FOR CREOSOTING TIMBER.

of the circumstances under which the finished product is to be used. Timber for piers, wharves and other structures in tropical waters demand processes and degrees of thoroughness of treatment that are unnecessary in the harbors of more temperate climates, which are, in turn, more exacting than land and fresh-water construction.

"It is as true as it is unfortunate, that, in the past—perhaps

at present—much creosoted work has fallen far below the reasonable expectation of the purchaser and user. As creosoting is neither a secret or patented process, nor are its operations complex, a close and systematic inspection of materials used at the place of manufacture

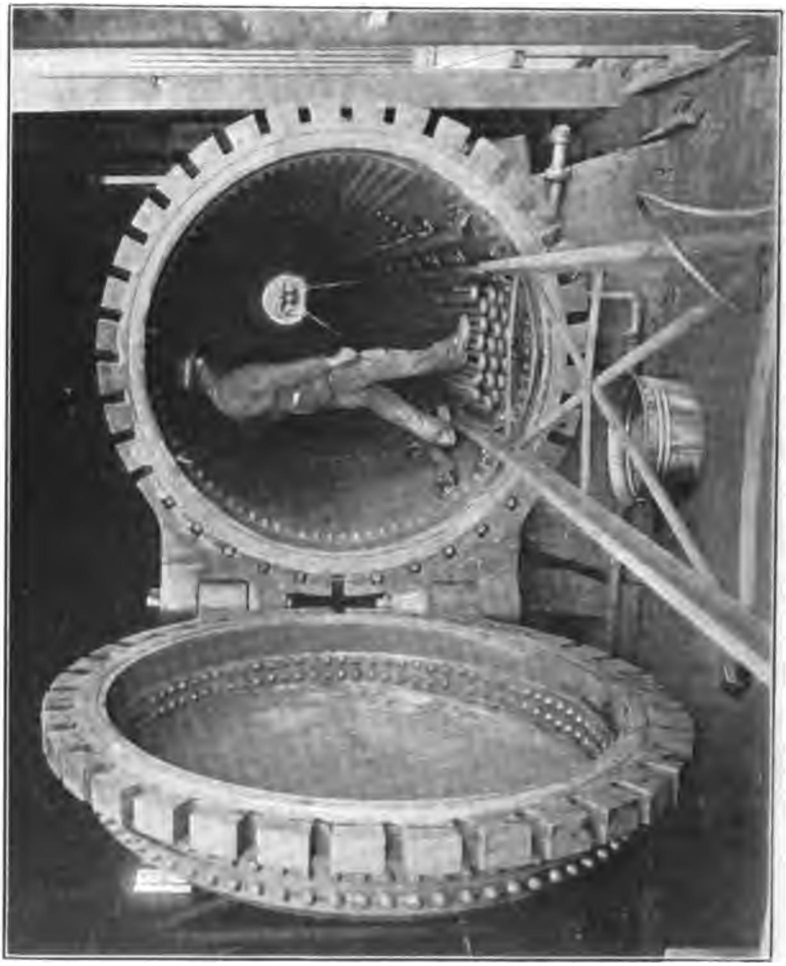


FIG. 399.—CREOSOTING RETORTS.

is all that is necessary for the buyer, and at the time that the creosoting is in progress."

The cost of creosoting varies of course with the size of piling and the size of timbers and with the location as well, but as a general average the cost of treating ordinary sized piling with 14 pounds of creosote per cubic foot will add about twenty-seven cents per lineal

foot to the price of the piling, and the cost of creosoted trestle timbers will range from \$25 to \$35 per thousand feet B.M. in addition to the price of the timber. Views in a modern creosoting plant are shown in Figs. 398 and 399, which plant is located near Seattle, Wash. The retorts are built strong enough to make it possible to fill specifications requiring large percentages of creosote to be put in fir timber, which is much harder to treat properly than other kinds.

The specification for the boiling process of treating timber with creosote, is given in full in Appendix X, by permission of F. D. Beal, Manager of the St. Helen's creosoting plant at Portland, Ore., and is a full description of this valuable method for hard-fiber timber.

Recent experiments on Douglas fir, both treated with creosote and without treatment, indicates that the creosoted timber is considerably weakened by treatment, the strength being reduced from 15 to 30 per cent.; the length of time taken for treatment and the amount of heat to which the timber is subjected being the largest factors affecting its strength. Air seasoning after treatment does not restore the strength of any appreciable degree.

The experiments are being carried out at the University of Washington by O. P. M. Goss of the Forestry Bureau, and the full report of the tests will soon be made public in a government bulletin.

CHAPTER XXIX

FOUNDATIONS FOR DAMS, SEAWALLS, AND BREAKWATERS

THE limited scope of this volume will not afford space for any discussion of the calculation, design, or construction of dams any farther than to discuss those matters which have particular reference to the foundations.

The first point to be considered in the construction of a dam is to have a practically unyielding foundation bed to prevent settling and the consequent leakage due to a poor foundation. This, of course, presupposes that the foundation on any material other than rock must be practically impervious, and if the material on which the dam is to be founded is rock, that it shall not be seamy and allow water to find its way underneath the masonry or superstructure of the dam. However, should the dam be founded upon hard gravel or solid earth, sheet-piling must be driven at the up-stream toe, to a sufficient depth to prevent the water with the head or pressure which will be realized when the dam is completed finding its way underneath in serious quantities. This sheet-piling may be replaced by a concrete cut-off toe wall carried to the requisite depth, or the cut-off or core wall may be incorporated as a part of the dam construction as indicated in Fig. 400. Where the dam is founded on porous or seamy rock, and this cannot be removed by going to any reasonable depth, a cut-off wall at the up-stream toe must be carried down to solid rock, or failing to find this, it will be necessary, if the rock is very seamy, to close the seams by forcing grout into them under pressure.

Another important matter to be taken care of is to carry the foundation far enough into the rock at the down-stream toe to prevent the scouring out of the foundation by the action of the water where the dam is of the over-flow type. This will very often cause the undermining of the down-stream toe from the back lash of the water, and in pervious foundation material it may be necessary to drive sheet-piling at the down-stream toe, or build a cut-off wall in its stead, although this is very often a dangerous piece of con-

struction, as it may confine the seepage finding its way underneath from the up-stream toe, and cause an uplift pressure on the dam, by reason of the free discharge being prevented.

The apron, Fig. 400, was originally carried a maximum distance of about 30 feet on the down-stream side on a hard gravel bottom, but as the back lash after a year or so of service undermined the apron to a considerable distance, the planking on the lower step of the apron was torn off and the rock and timber grouted in with wet concrete, and the apron carried about 20 feet farther down-stream. After being in service for a period of five or six years no further trouble had been had from this source. This dam had two spill-way sec-

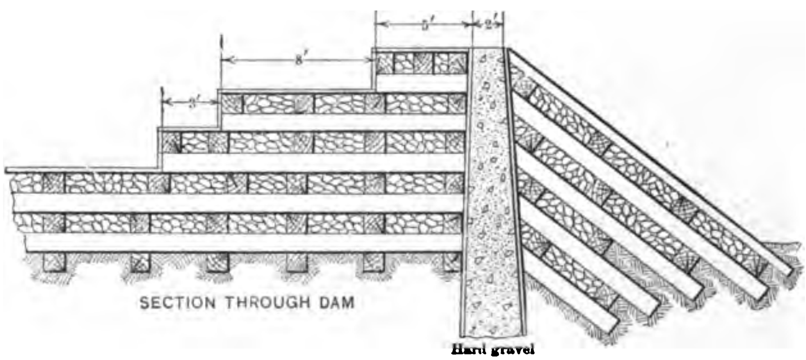


FIG. 400.—YAKIMA DAM SECTION.

tions and three sections with Chanoine wickets operated from a bridge as shown in Fig. 401.

In every dam the structure must be carried into the bank on either side, so as to prevent the water under pressure from finding its way around the end. The dam in Fig. 401 had one end carried into solid rock, while the other end was carried well into the gravel bank, and two rows of round piling driven close together to quite a distance farther on shore to make certain that no water would find its way around the end.

On account of some very dangerous floods it was, however, found advisable, after a year or so, to extend the concrete cut-off wall entirely across the canyon, to where rock bottom was again encountered, and to build a reinforced concrete dam connecting from that point across a narrow rocky wash to the far rock bluff. These things should have been foreseen and provided for in the original plan.

The dam for the Tacoma Water System on Green River (Figs. 402, 403 and 404) constructed by the author, was supposed to have

been located on firm rock, but upon pumping out the coffer-dam the upper portion proved to be very seamy, and it was found necessary to carry the entire dam from 3 to 6 feet lower or into the solid rock, and the cut-off walls were both omitted, although the lower cut-off wall was evidently a mistake in the original design. The first season's floods demonstrated the additional fact that the apron had not been carried far enough down-stream, as the rock became soft on exposure, and the bottom scoured out into holes or pockets from 1 to 6 or 7 feet in depth, making it necessary to extend the apron 20 to 30 feet farther down-stream, due to oversight in making original plans. The abutments at both ends of the dam were carried into the solid rock so that no water could find its way around the ends.

FIG. 401.—YAKIMA CHANOINE WICKET CRIB DAM.

The necessity will sometimes arise for putting in the foundation of a dam where the best thing possible to be done must for reasons be abandoned, and the construction carried on using the best judgment of the engineers connected with the work. In a case of this kind where the author acted as consulting engineer on the construction of a dam in the rapids just above the 270 feet drop of Snoqualmie Falls in Washington, the endeavor had been made for some months to construct a dam, and late in the summer it was determined to be a case of absolute necessity to get in a dam before the Fall floods, in order to give the necessary head of water to operate a hydro-electric plant. The fall of the water in the rapids was sufficient, so that by using a semicircle of sandbags around a section of the dam in the stream, the water would drop away below, and leave the bed-

FIG. 402.—GREEN RIVER DAM.

FIG. 403.—GREEN RIVER COFFER-DAM.

rock practically dry. To get the dam completed in a little over thirty days, it was impossible to do any more than to clean off the bed-rock, leaving the large boulders to be incorporated in the concrete; and to properly anchor the dam, large enough holes were drilled into the bed-rock with air burleys to set in short sections of railroad rail every few feet to anchor the dam against sliding. A cable-way was stretched across the stream on the axis of the dam (Fig. 405), and a timber frame of the section of the dam con-

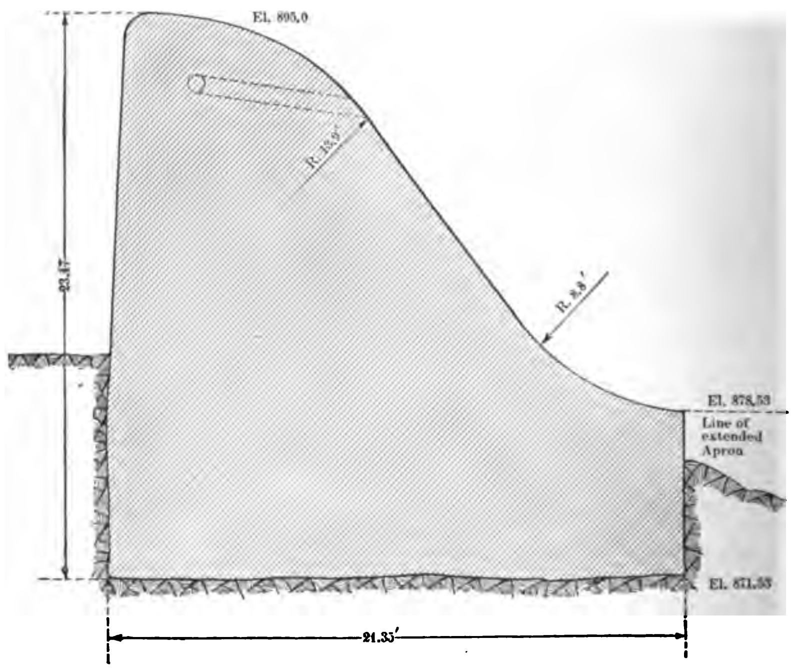


FIG. 404.—GREEN RIVER DAM SECTION

structed, and this planked with 6-inch timber drift-bolted in place, excepting the top plank, which was left off until the form was filled up with concrete. Another section could then be built in a similar manner, leaving a 5- or 6-foot space between the sections for a spillway, these sections being closed up and filled the last thing by battening the plank together and utilizing the force of the water to press them into place, after which the concrete was poured. In this way the dam was constructed in practically thirty days, by working day and night crews, and it has stood for about fourteen years with

no serious repairs except to put a block of concrete in one deep hole below the dam where some scour had occurred.

Some of the high dams are of the overflow type, and the section of the down-stream face, and the design of the apron becomes a very

FIG. 405.—SNOQUALMIE RIVER DAM.

important matter, although most of the high dams are provided with spillways to carry the flood waters without endangering the down-stream toe of the masonry.

The cross-section of the Croton Dam, founded on solid rock, is shown in Fig. 406, showing as well the anchor pits, 10 feet in width

and 6 feet deep, which were put in to prevent the dam from sliding upon the base.

The cross-section of a dam at Grenoble, France, across the River Drac, is shown in Fig. 407, and represents a type of apron that will

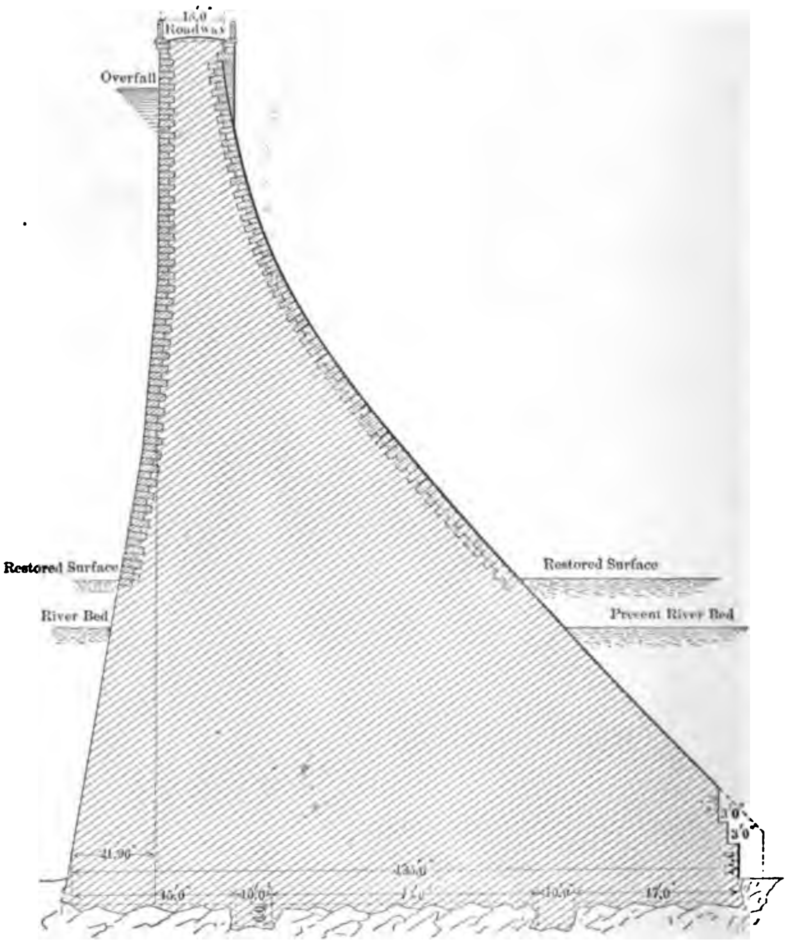


FIG. 406.—CROTON DAM FOUNDATION.

carry the water far enough down-stream and discharge it in such a way as to almost entirely prevent back lash.

The foundation of the Assuan Dam is shown in Fig. 408, showing how the rock was excavated in notches to prevent sliding.

The section of the Spaulding Dam in Nevada County, California,

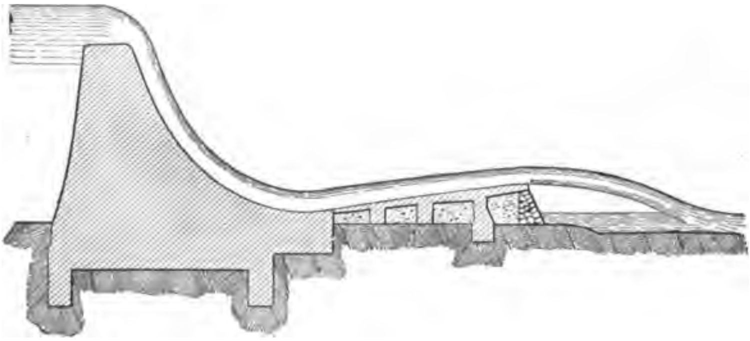
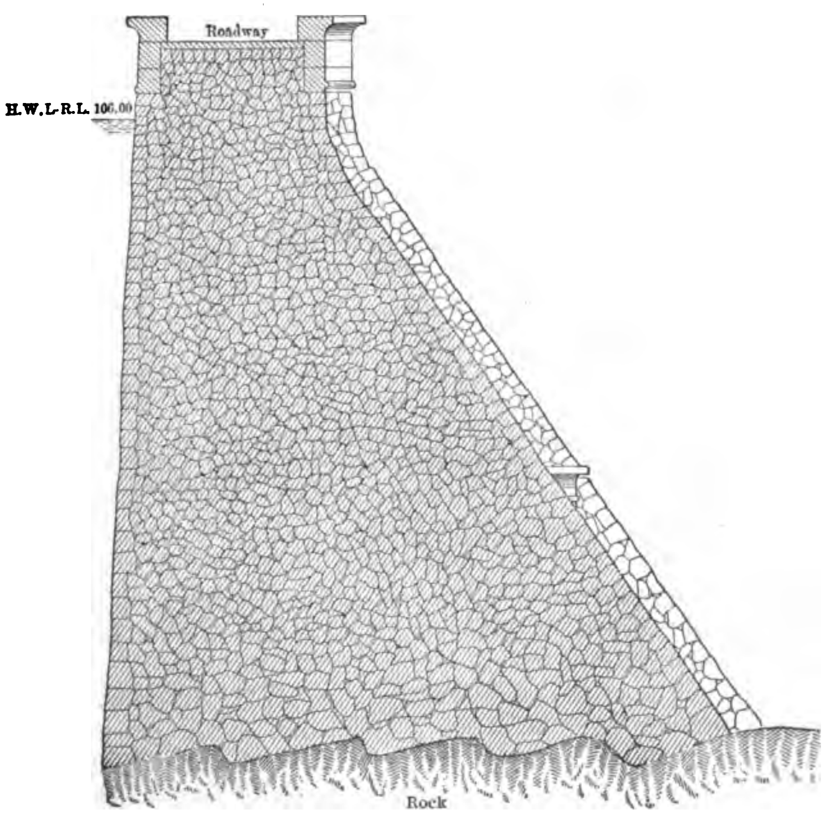


FIG. 407.—DAM FOUNDATION AND APRON, GRENOBLE, FRANCE.



CROSS SECTION OF DAM SHOWING ABUTMENT PIER

FIG. 408.—ASSUAN DAM FOUNDATION

and the profile of the canyon are shown in Figs. 409 and 410. This shows the character of the foundation, and the following discussion of it as relating to the foundation proper is taken from the *Engineering Record* of August 9, 1913:

The new dam is being constructed across the canyon of the south fork of the Yuba, about 2200 feet down-stream from the present dam. At the dam site the geological formation is of granite and diabase, affording an excellent foundation.

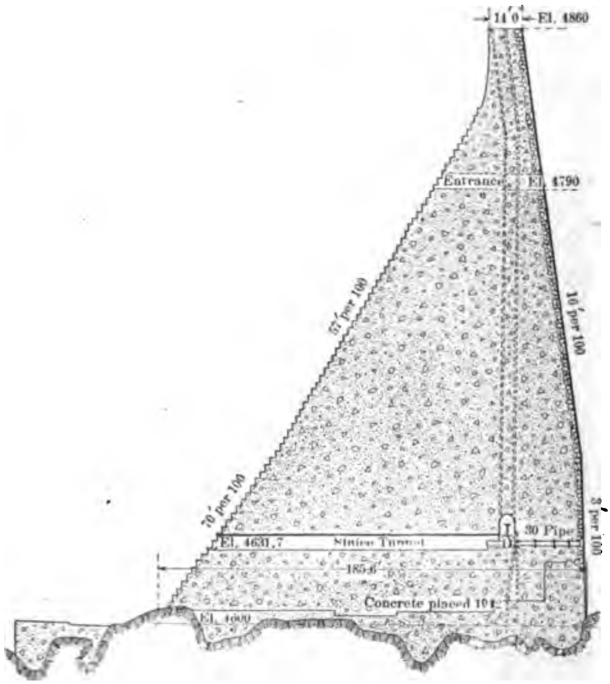


FIG. 409.—SPAULDING DAM SECTION.

The dam itself is of cyclopean masonry. Although designed as a straight, gravity-section dam, it is arched in plan to provide an additional factor of safety.

This arch is of the type developed by Mr. L. Jorgensen, of the engineering firm of F. G. Baum & Company, of San Francisco, the main feature of which is that the angle subtended by the arc under pressure is kept as nearly constant as possible by varying the radius of the up-stream side at different elevations. This is considered a very desirable condition, since arch action in the dam can thus be utilized to a maximum degree at every elevation, which tends greatly

toward economy of section. When an ordinary arched dam is located in a canyon, which is narrow at the bottom and much wider at the crest, practically no arch action takes place for a considerable distance vertically above the toe. This is obvious, for the reason that near the bottom of the dam the angle subtended by the arc under pressure is so small that the arc is practically a straight line. Therefore this portion of the dam acts as a beam rather than as an arch. By using a shorter radius at the bottom, however, arch action can be utilized where it is most essential, namely, near the base of the dam, where the pressure is a maximum.

Accumulation of water between the dam and its foundation will be prevented by 6-inch weep holes, extending from the foundation vertically to the inspection gallery. A similar set of weepers is pro-

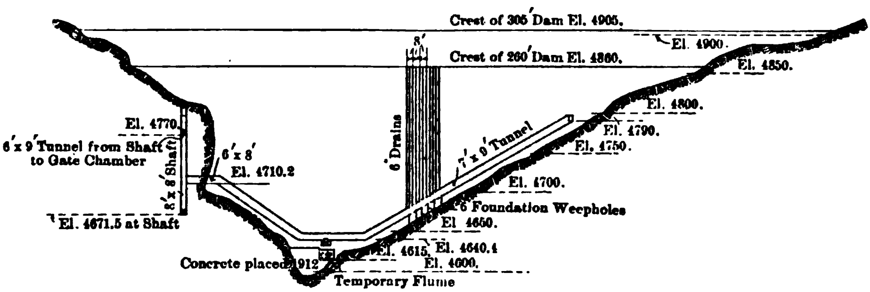


FIG. 410.—SPAULDING DAM, PROFILE ACROSS CANYON.

vided above the gallery, so as to collect possible seepage in the body of the dam.

Water will be drawn from the reservoir through two 6×9-foot intake tunnels opening in the granite canyon side about 12 feet from the face of the dam. The elevations of the tunnel inverts above the level of the toe of the dam are 71.5 and 170 feet respectively. Each tunnel is controlled by a separate gate valve, so that water may be drawn from either level. From these gates the tunnels lead to a shaft 8 feet square, and from this shaft a single concrete-lined tunnel 8 feet 8 inches in diameter in the clear and 4456 feet long, with a fall of 7 feet per mile, leads to the main canal.

There will be two concrete spillways, entirely distinct from the dam, located in adjoining canyons, as shown in the plan.

During the construction the flow of the South Yuba River is carried through the dam in two flumes, each 5 feet square. All material is handled by the gravity system. The company has built a spur track from Smart to a point directly above the dam site, where

the gravel and sand are dumped into bunkers below the cars and from there are brought to the mixers by horizontal belt conveyers, each operated by a 50-horse-power motor. These belts are 18 and 24 inches wide and travel at the rate of 300 feet per minute. The cement is unloaded from the cars to a similar belt traveling at the rate of 100 feet per minute. Gravel, sand and cement are thus conveyed to four 1-cubic-yard hoppers, each of which is located directly over a 1-cubic-yard Smith mixer. The hoppers also have a capacity of 1 cubic yard, so that each batch contains exactly the same amount of material. The proportions of sand and gravel are accurately gaged, as the belts pass over a movable tripper, which directs the material into any one of the four hoppers desired. A 1–2–4 mix is used. Each mixer is driven by a 10-horse-power motor. The combined capacity of the four mixers is 2000 cubic yards in twenty-four hours.

The mixers empty their contents into a chute $2\frac{1}{2}$ feet deep and 2 feet wide which zigzags down the side of the canyon to the dam, each section having an incline of about 25°. The lower sections are movable, so that concrete can be placed wherever wanted. This system will be employed until the dam reaches a height of about 225 feet, when an elevator must be erected and the concrete raised from the mixers to a sufficient height to enable it to be placed by gravity even at the highest point.

After the concrete has been deposited and while it is still plastic large pudding stones are dropped into it. These are handled by three derricks, which will be raised from time to time as construction progresses and the height of the dam increases.

The hollow reinforced concrete dams which are being constructed in many locations are of such a type that no up-lift pressure will occur, although the leakage underneath the up-stream toe sometimes becomes a very serious matter, and in one case that the author has in mind, seepage was so great that it became necessary to grout the seamy rock at great expense. Further information and data on the subject of dam foundations will be found in Wegmann's "Design and Construction of Dams," 6th Edition, 1911.

The investigations carried out by the author in regard to the construction of seawalls for the Seattle Harbor Commission disclosed the fact that there have been constructed in the various ports of the world almost as many different ideas and designs of walls as there have been engineers engaged upon their construction.

The most simple ones are merely timber bulkheads, very often of the type shown in Fig. 257; many are of pile and brush construction,

as described in Chapter XVIII; while many have timber sheet-piling supported by guide-piles and waling, and anchored back to anchor piles or "dead-men." This type is sometimes constructed of creosoted timber and piling, and will, if properly designed, last for twelve or fifteen years.

Where the shore is protected with riprap, as shown in Fig. 335, and it is desired to have vessels laid up along the shore, a wharf or quay can be built parallel to the shore by driving the piling before placing the riprap, or else driving the innermost row of piling through the toe of the riprap and spanning the distance from the piling to the shore with timber or hog-chain girders. Such seawalls of timber, brush or riprap can be constructed at costs ranging all the way from $5 per lineal foot to $12 or $15 per lineal foot for ordinary heights, the engineer, of course, making an estimate of each special case, based upon the cost of materials and labor for the particular locality.

The construction of masonry seawalls is one which should be gone into very carefully for each particular case, bearing in mind that the water is liable to get underneath the wall, and very often saturate the material back of the wall so that it will cause both an uplift and a very greatly increased pressure from the landward side. This, of course, would make the danger much greater for sliding upon the base.

The construction of a wall at the Puget Sound Navy Yard (Fig. 411) 40 feet in height, was carried out by the author by driving sheet-piling around the outside conforming to the batter of the wall, down into the cemented gravel, and after properly bracing it, the dredging was done inside of the sheeting with an orange-peel bucket down to the required depth, and the concrete up to low water deposited with a *trémie*. For walls of this type investigated by the author, including piling under the base in softer material, with the concrete 42 feet in height, the estimated cost was about $175 per lineal foot at Seattle. Where only the footing course about 8 feet in thickness was deposited through a *trémie*, and the balance pumped so as to deposit the concrete in the dry, the cost was found to be about $195 per lineal foot, there being only the height of 12 feet, from the top of the base to mean low water, requiring much pumping; above this portion very little pumping would have to be done. Among other designs there was a quay wharf supported on reinforced concrete piles, with the fill held in place by reinforced concrete sheet-piling, and this was estimated to cost, for practically the same depth of water alongside, not over $100 per lineal foot.

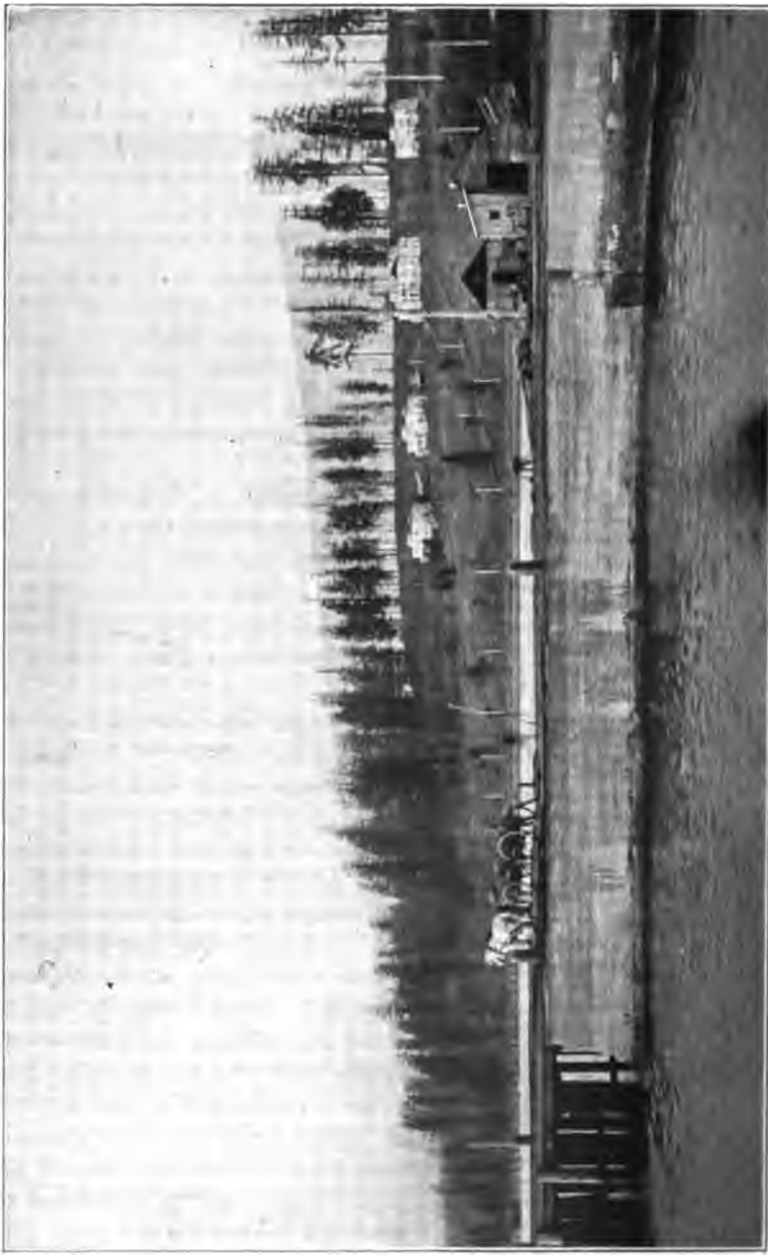
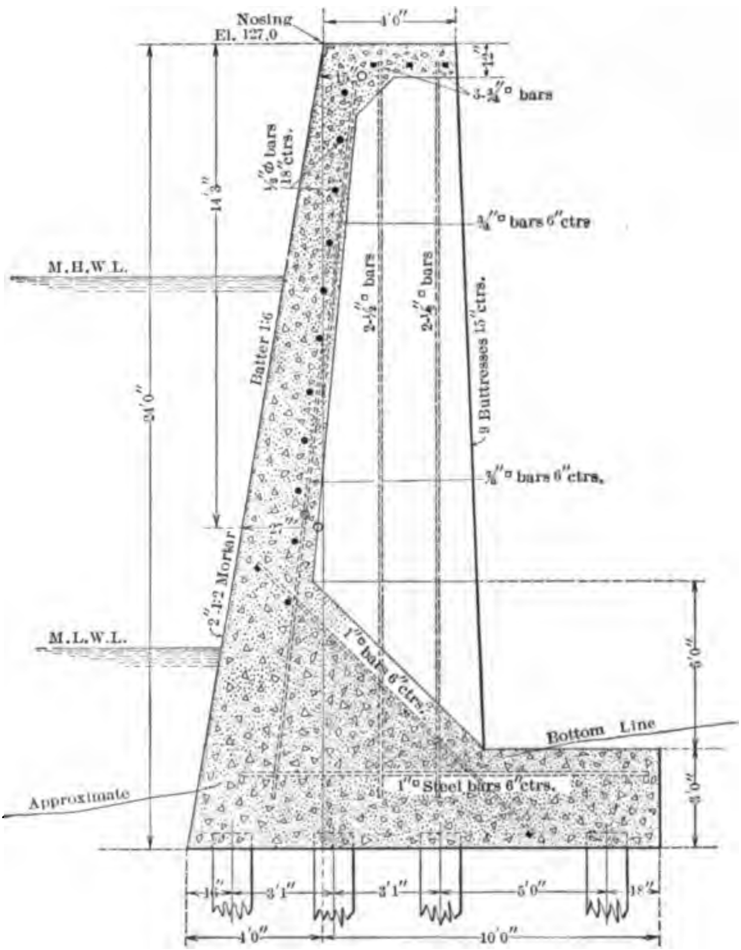


FIG. 411.—PUGET SOUND NAVY YARD.

The later seawalls constructed at the Puget Sound Navy Yard (Fig. 412) are of reinforced concrete and range from 9 feet in height to 24 feet in height of the sections shown, with the supporting piling spaced closer near the outside face to take up the increased load

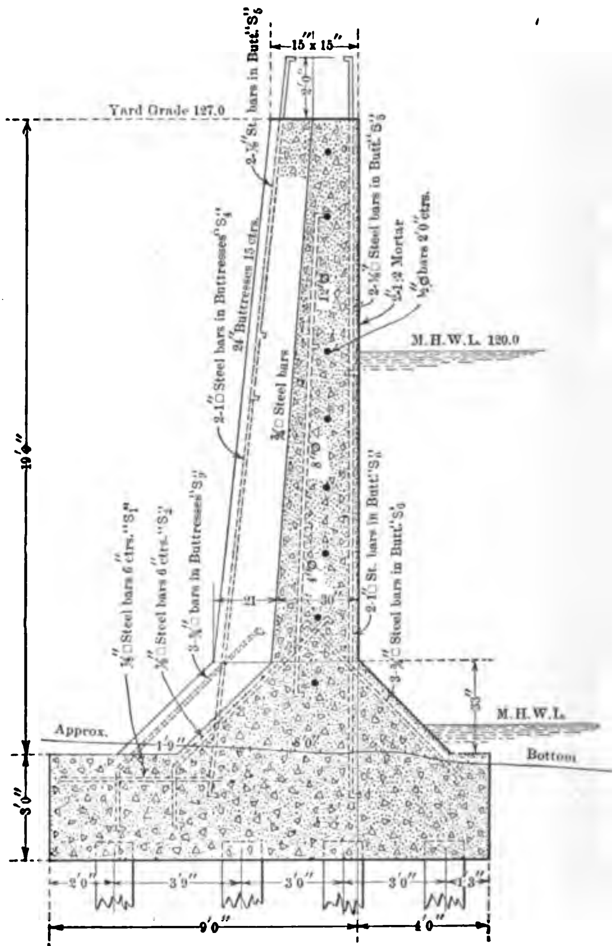


24 FT. SECTION
Rows of piles 3'9" ctrs,

FIG. 412.—PUGET SOUND NAVY YARD, SEA WALL.

due to the overturning moment. These walls are of a type where the vertical component of the earth filling is utilized to add to the stability of the wall. The nosing of the wall is of cast iron anchored to the concrete as shown in Fig. 412 (*b*). The itemized cost of walls

of this type is given in Chapter XXXIII, and while some of the older walls were constructed of the gravity type, reinforced construction undoubtedly represents a saving of from 25 to 30 per cent.



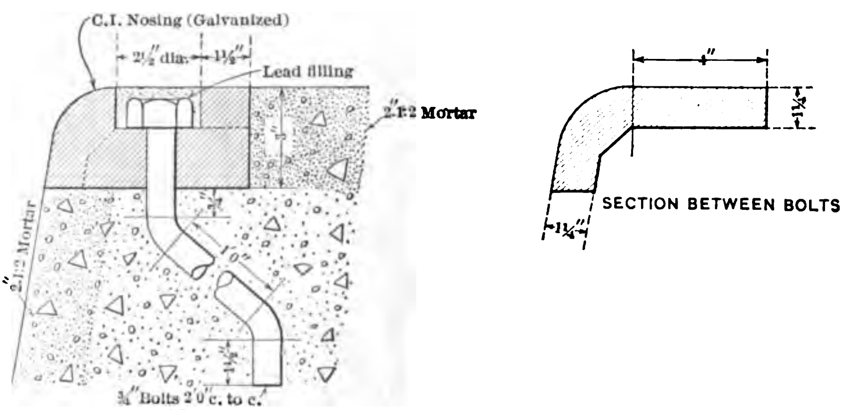
Rows of piles 7½ ft. ctrs.
22 FT. SECTION

FIG. 412(a).—REINFORCED SEAWALL, PUGET SOUND NAVY YARD.

The masonry seawalls constructed in New York Harbor comprise over a score of different designs. The type shown in Fig. 413 consists of a platform supported by vertical piling with 12×12 sheet-piling at the shore side of the platform to retain the fill, and brace

piles as shown to take up the thrust. On top of this platform is constructed a concrete seawall about 9 feet 6 inches in height, with a base of 7 feet, to retain the earth filling which carries the pavement. Outside of this wall timber fender-piles are driven and fastened as shown. Another type of wall founded or resting directly on a pile foundation is shown in Fig. 414, the piling being filled around with riprap, and both faces of the wall being riprapped as shown.

The wall shown in Fig. 415 is founded upon rock and is of concrete construction with granite facing above low water. The inside of the wall is stepped, thus taking advantage of the vertical loading from the earth fill. The type of wall used at Cedar Street (Fig. 416) is founded upon rock, the leveling off of the foundation bed having



SECTION AT BOLTS

DETAIL OF COPING(*a*)

FIG. 412(*b*).—SEAWALL NOSING, NAVY YARD.

been accomplished by using concrete in bags. The wall at the Battery (Fig. 417) is founded on a base of riprap on the rock as shown.

On account of the great thrust or overturning effect of the fill, many of the New York seawalls have been constructed like the Canal Street section (Fig. 418) with a relieving platform as shown, relieving the wall from the greater portion of the earth thrust.

The minimum cost of the rock-bottom type for the wall proper has been about \$160 per lineal feet, the maximum cost about \$310 per lineal foot, and the average cost being about \$260 per lineal foot. The average total cost has been slightly under \$300 per lineal foot. The minimum cost for the hard bottom type has been about \$125 per lineal foot for the wall proper, the maximum cost about

$135 per lineal foot, with an average of about $130 per lineal foot; the total average cost of this type being slightly under $240 per lineal foot. The minimum cost for the walls proper for the relieving platform type, has been under $100 per lineal foot, the maximum cost for the wall proper, slightly under $140 per lineal foot, with an

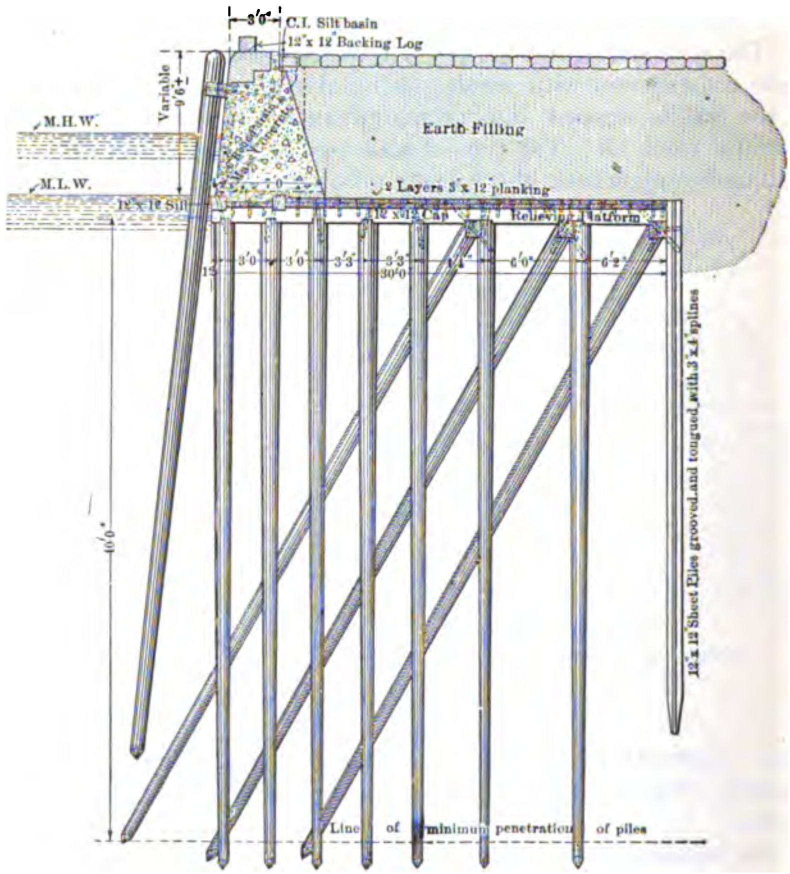
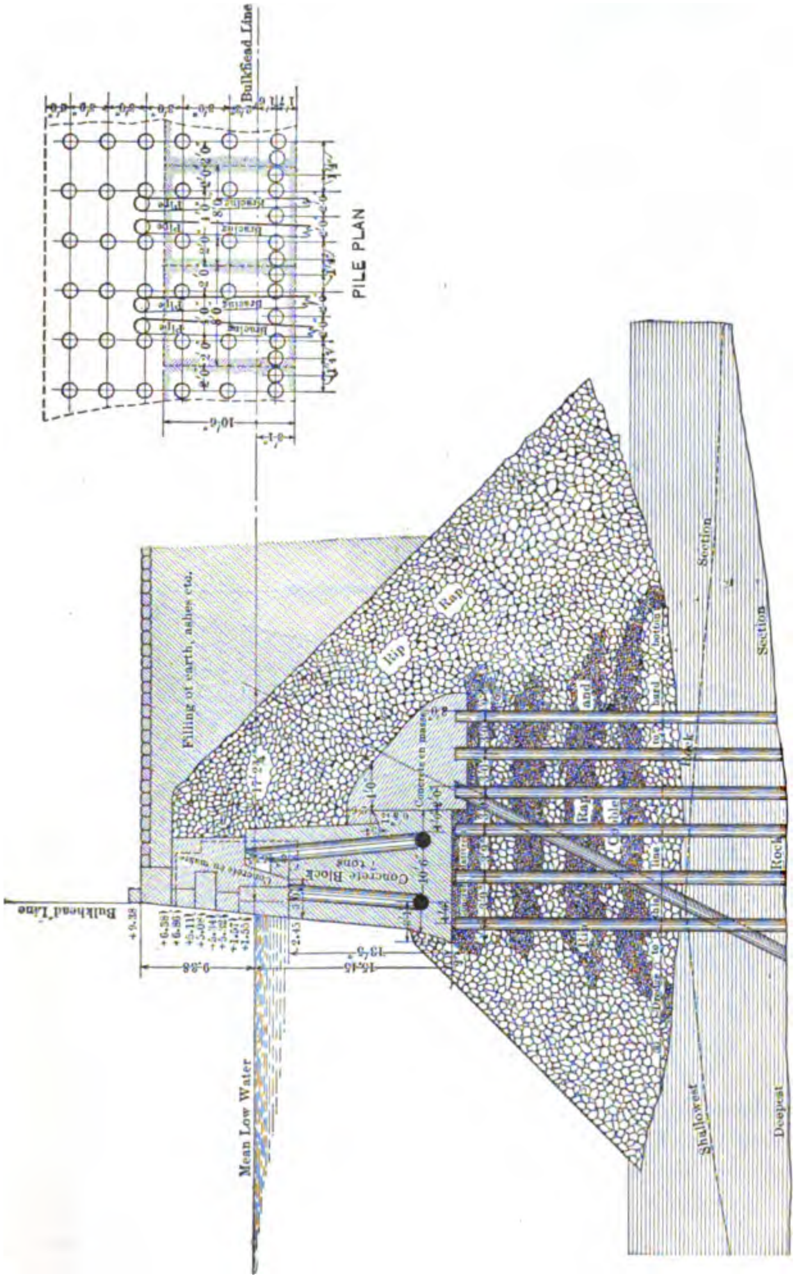


FIG. 413.—NEW YORK SEAWALL, PILES AND PLATFORM.

average of about $110 per lineal foot. The details of these costs per lineal foot are set forth in the table on p. 604.

"One of the latest pieces of the extensive municipal seawall construction work in Hamburg, Germany, was done last year at the slip alongside the yard of Blohm & Voss in order to make this slip available for steamers of the *Imperator* class during the finishing



RECTOR ST. SECTION

Fig. 414.—New York Seawall Pile Foundation.

TABLE LXVI.—COST OF NEW YORK SEAWALLS.

Item.	Rock Bottom. Type.			Hard or Firm Bottom. Type.			Relieving Platform. Type.		
	Min.	Max.	Av.	Min.	Max.	Av.	Min.	Max.	Av.
Dredging..........	$6.60	$107.00	$30.00	$11.00	$44.00	$32.00	$13.00	$62.00	$30.00
Riprap and cobble..	4.50	12.50	10.40	12.50	19.50	16.00	24.00	84.00	44.00
Piling and timber work...........	....	.....	....	49.00	62.00	56.50	72.00	139.00	89.00
Concrete and granite	156.00	309.00	254.0	125.00	133.00	129.50	88.00	139.00	109.00

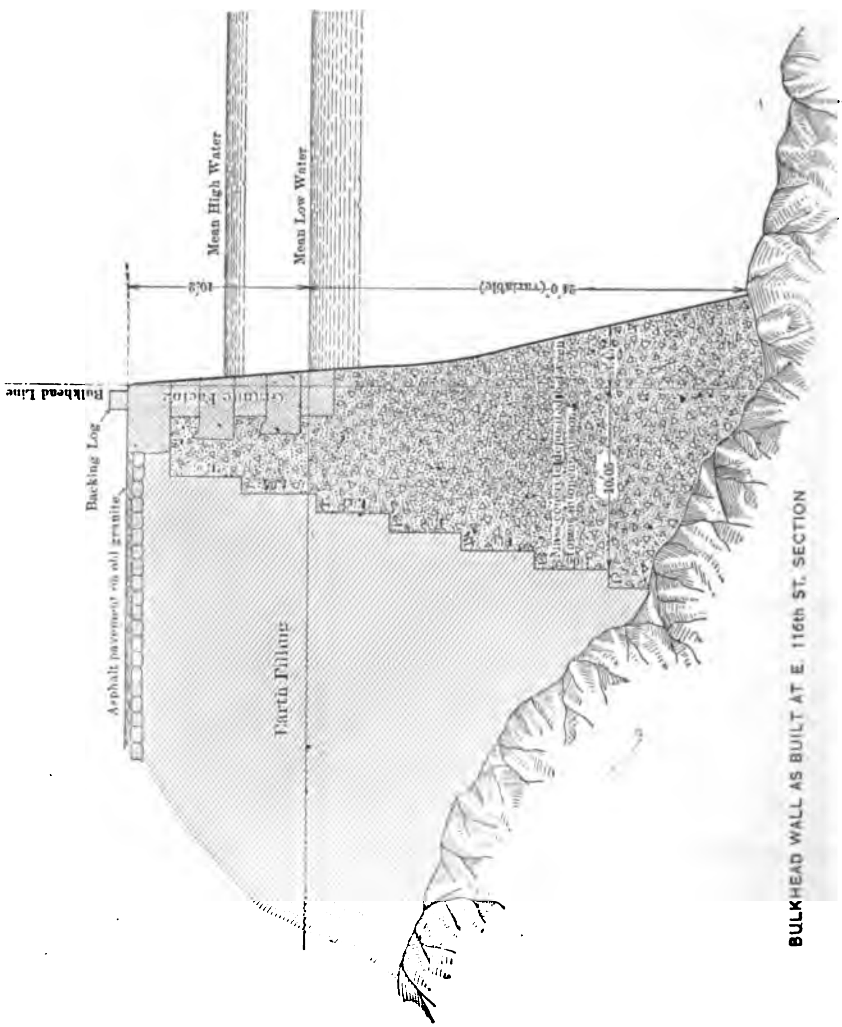


FIG. 415.—NEW YORK SEAWALL ON ROCK.

work after launching. Since the improvement, the hull of the giant *Vaterland* has occupied the slip. The slip had to be deepened from 25-foot depth to 33-foot depth at mean low water. Partly for this reason and partly because the existing quay wall was of unsuitable kind, the entire quay wall had to be rebuilt. At the same time the shipbuilding firm put up a 275-ton tower crane on the quay, its foundation being independent of that of the wall.

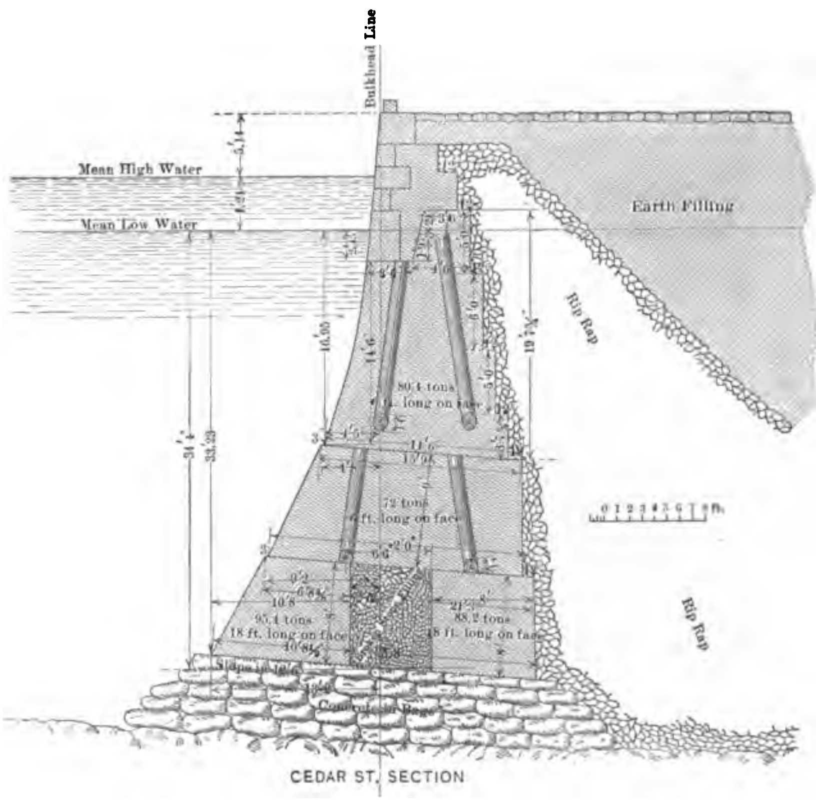
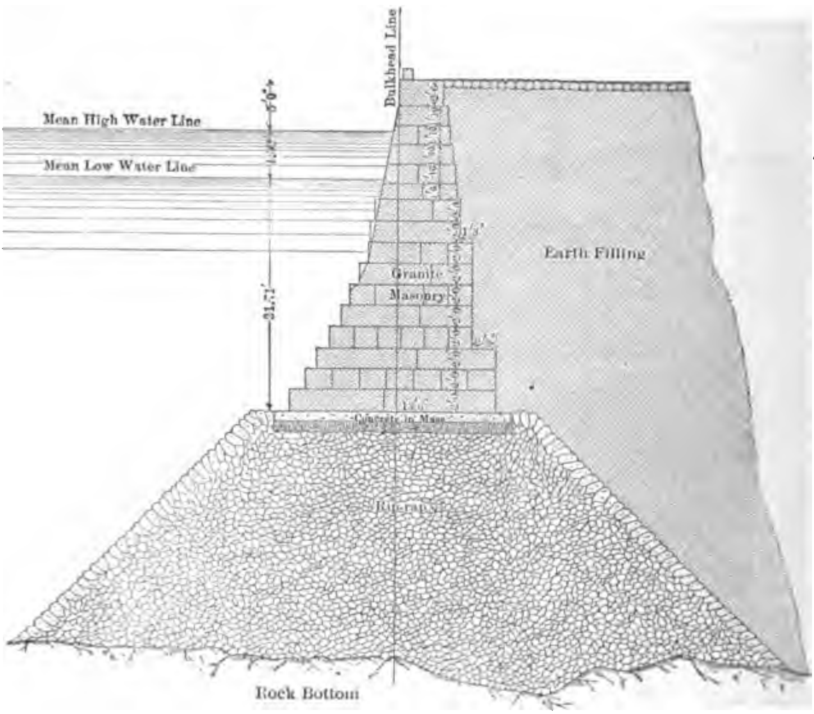


FIG. 416.—NEW YORK SEA WALL. CONCRETE BAGS ON ROCK.

"The wall is shown by the drawings, Fig. 429. A small sketch of the old wall, indicating the relative positions of old and new walls, is included in the drawing. The old wall consisted of masonry piers 82 feet apart and intermediate sections of sloping bank paved with cement blocks, over a foot wall of braced piles set close. The new wall is a concrete base slab and front wall for retaining the fill, resting on pile and grillage foundation. The masonry work was

done in the open, and, therefore, the top of grillage is fixed at 1.7 feet above low water.

"The extreme tidal range at Hamburg is from 2.35 feet up to 28.5 feet above Hamburg Datum, but the ordinary range is only $6\frac{1}{2}$ feet, from El. 9.8 feet to El. 16.4 feet. The desired low-water depth is 10 m., requiring the slip to be deepened to El. -23 feet (-7.0 m.). The mean high-water depth, therefore, is 39.4 feet. As will be seen from the sketch, the elevation of top of quay wall

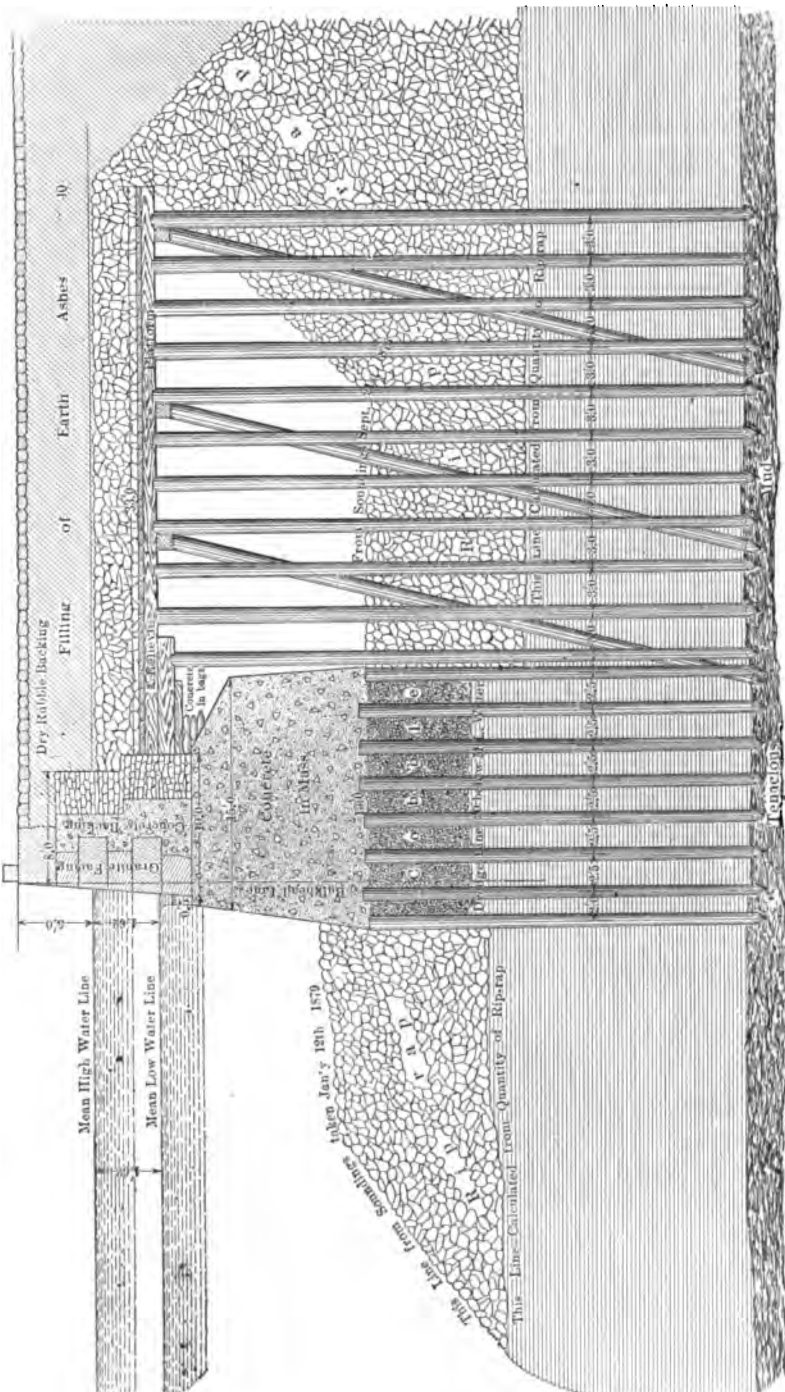


BATTERY SECTION. "A"

FIG. 417.—NEW YORK SEA WALL. RIPRAP ON ROCK.

(9.2 m.) is just above extreme high tide and gives a freeboard of about 20 feet at mean low tide.

"The use of pile-and-grillage substructure is the standard for quay-wall work in Hamburg, this practice having prevailed now for a considerable time past. The piles are 16-inch butt diameter, and average about $4\frac{1}{2}$ feet spacing. They are capped by longitudinal 8×10-inch caps, and over these is an $8\times9\frac{1}{2}$-inch cross-sill over each transverse pile row. The longitudinal caps are notched into the pile



CANAL ST. SECTION, "A"

FIG. 418.—NEW YORK SEA WALL RELIEVING PLATFORM.

top and bolted through. A 2-inch plank floor is laid over the caps between cross-sills, and on this the concrete and stone facing work is erected. The timber is all fir, except that the cap of the front row of piles is oak.

"The concrete of the body of the wall is 1:8 mixture (cement and gravel). The front of the wall is faced with blocks of columnar basalt, which are quite regular in shape and make a neat and very durable facing. This is also a standard detail in Hamburg quay-wall construction.

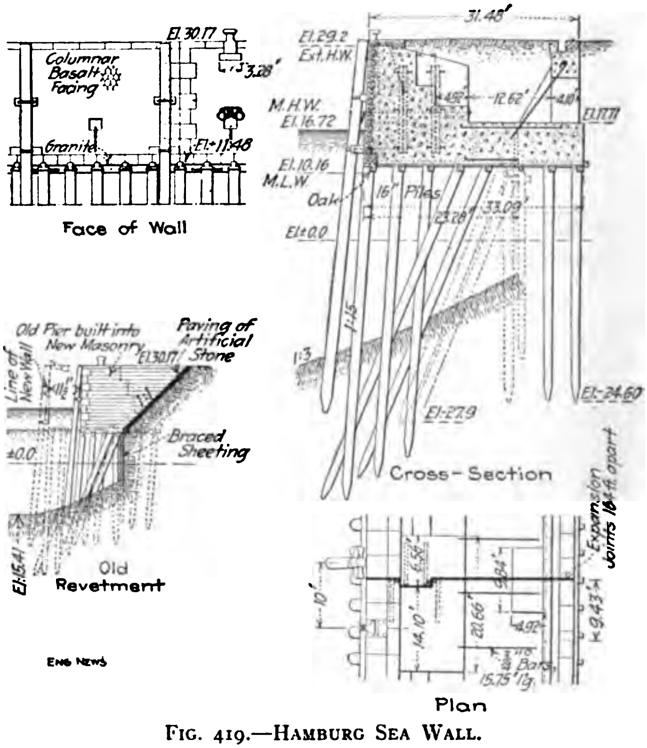


Fig. 419.—Hamburg Sea Wall.

"At the rear of the base slab there is a back wall for carrying the rear truck of a gantry crane running along the quay; in fact, provision for this crane track was the reason for making the grillage and base slab as wide as shown. Longitudinally the crane has a wheel base of 79 feet. Its total loaded weight will be 1100 tons. The longitudinal slot in the top of the back wall, seen in the cross-section, is the conduit for the electric cables supplying power to the crane.

"The cost of the quay wall was approximately $460 per lineal meter, or $140 per lineal foot. This is lower than the normal cost of such a structure by about $45 per lineal foot, on account of the presence of the old revetment.

"To give some basis of judging of the above cost figures, it is noted that the cost of the foundation piles in place (i.e., driven) is as follows:

Av. diam. of Pile.	Cost in Place per Lin. Ft.
20 in.	$0.95–1.10
18 in.	0.80–1.00
16 in.	0.75–0.95
14 in.	0.60–0.75

FIG. 420.—HAMBURG TOWER CRANE.

"The concrete (1 to 8 mixture) cost $3 to $3.50 per cubic yard. The coping granite cost about $33 to $37 per cubic yard, in place. The average rate of pay of concrete workers is about 19 cents per hour; masons and carpenters are paid 6 to 7 cents more.

"The 275-ton fixed tower crane already mentioned was shown in an outline sketch in *Engineering News* of Oct. 31, 1912. Its appearance is reproduced more strikingly by the view Fig. 420 herewith. The long or front arm of the crane can tilt upward, being operated by a pair of pulling screws attached to links in the line of the top chord. The load of 275 tons (250 metric tons) can be carried at a radius of 105 feet from the center. The small locomotive

crane running on top of the arm has a capacity of 22 tons at half projection, or 11 tons at full projection of 52½ feet from its own center, which makes the extreme reach 229½ feet from the center of the tower. The top chord of the main crane is about 180 feet above quay level. The drive of the crane is wholly electric, the control being of the Leonard type." The above account is taken from the *Engineering News*.

The most notable example of quay-wall construction in recent years was the building of over 6500 lineal feet of wall in connection with a new port project at Antwerp, Belgium, on the River Scheldt (Fig. 421). This wall (Fig. 422 (*a*)) had a base of 31.2 feet, the base proper being about 10 feet thick, with the total height of wall of about 56 feet. This wall had to be constructed in the river and was founded by compressed air through the silt down to the underlying hard bottom. Inasmuch as the wall would have to be subjected to variable pressures during the progress of the dredging work and the backfilling, and later permanently due to variations in the tide, the work had to be carried out in a way that would leave no uncertainty as to the character of the foundation bed. There were eleven different bids received on this work in April, 1907, representing some thirty different propositions, and the bid of Hersent & Sons of Paris was accepted, amounting to the sum of 10,900,000 francs, as the proposition was finally modified and decided upon, or over $2,000,000 for the 2000 meters of wall. The bids ranged from $245 per lineal foot to about $630 per lineal foot, while the bid accepted was for approximately $300 per lineal foot, and including the total expense up to the time of the completion of the work, the ultimate total cost of everything, including fill, amounted to about $480 per lineal foot.

The design of the wall as it was constructed may well be said to be a beautiful one, and also of the very best in its engineering features. This was, however, a modification of the cross-section submitted by Hersent & Sons, which had a base of only 8.7 meters as against 9.5 meters as built, or slightly over 9 per cent. of an increase, and greatly increasing the stability against rotation.

The curved batter of the face of the wall and the stone facing from below low water to the stone coping add much to its appearance. The conduit or gallery in the wall near the top is provided for placing pipes and wiring for the port, thus making it unnecessary to dig up back of the wall for laying pipes or wires. This conduit is about 4 feet wide by 6.6 feet high and has a ledge about 20 inches wide on which to lay pipe and the like.

The researches carried out in regard to the bearing capacity of

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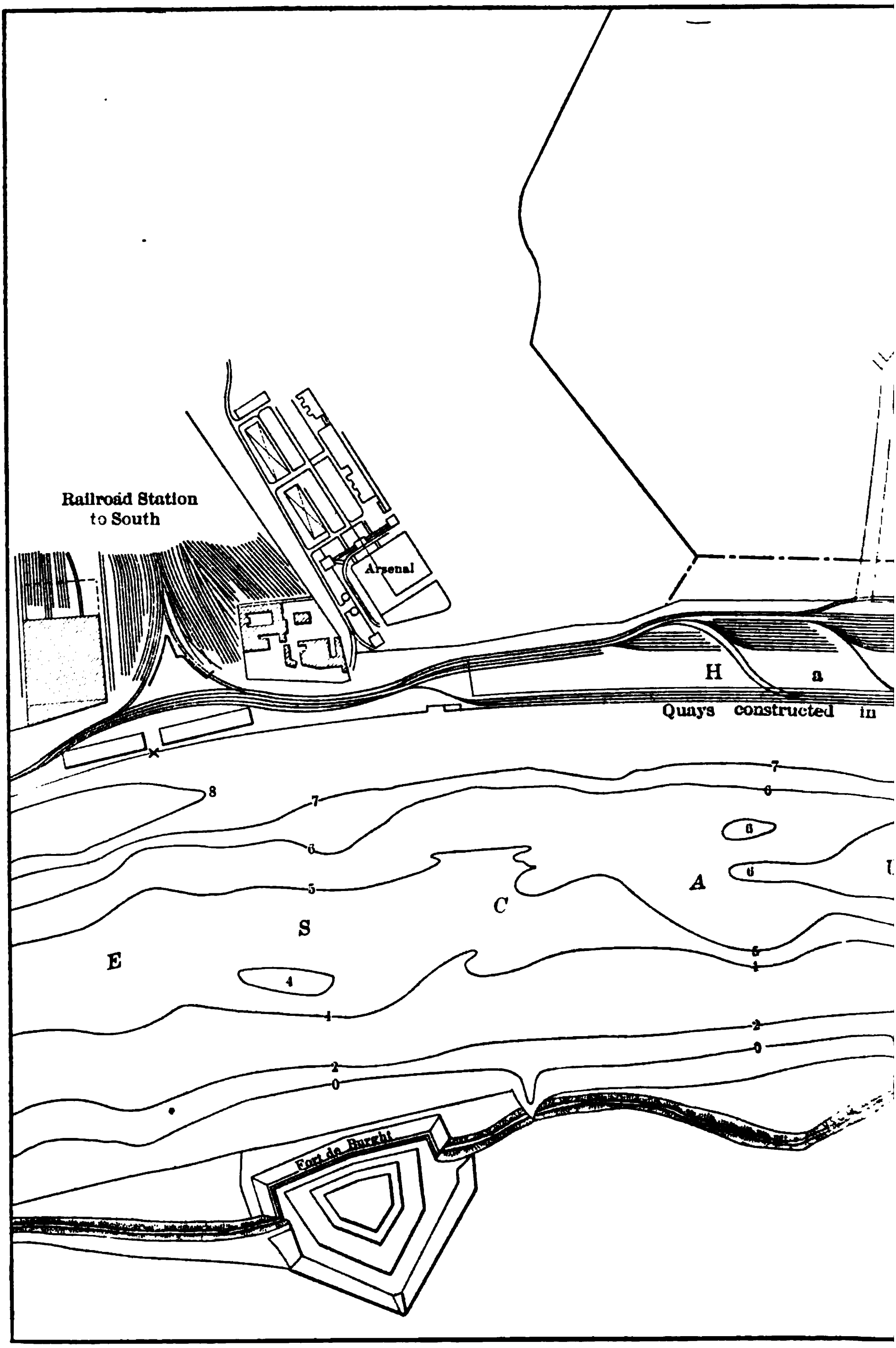
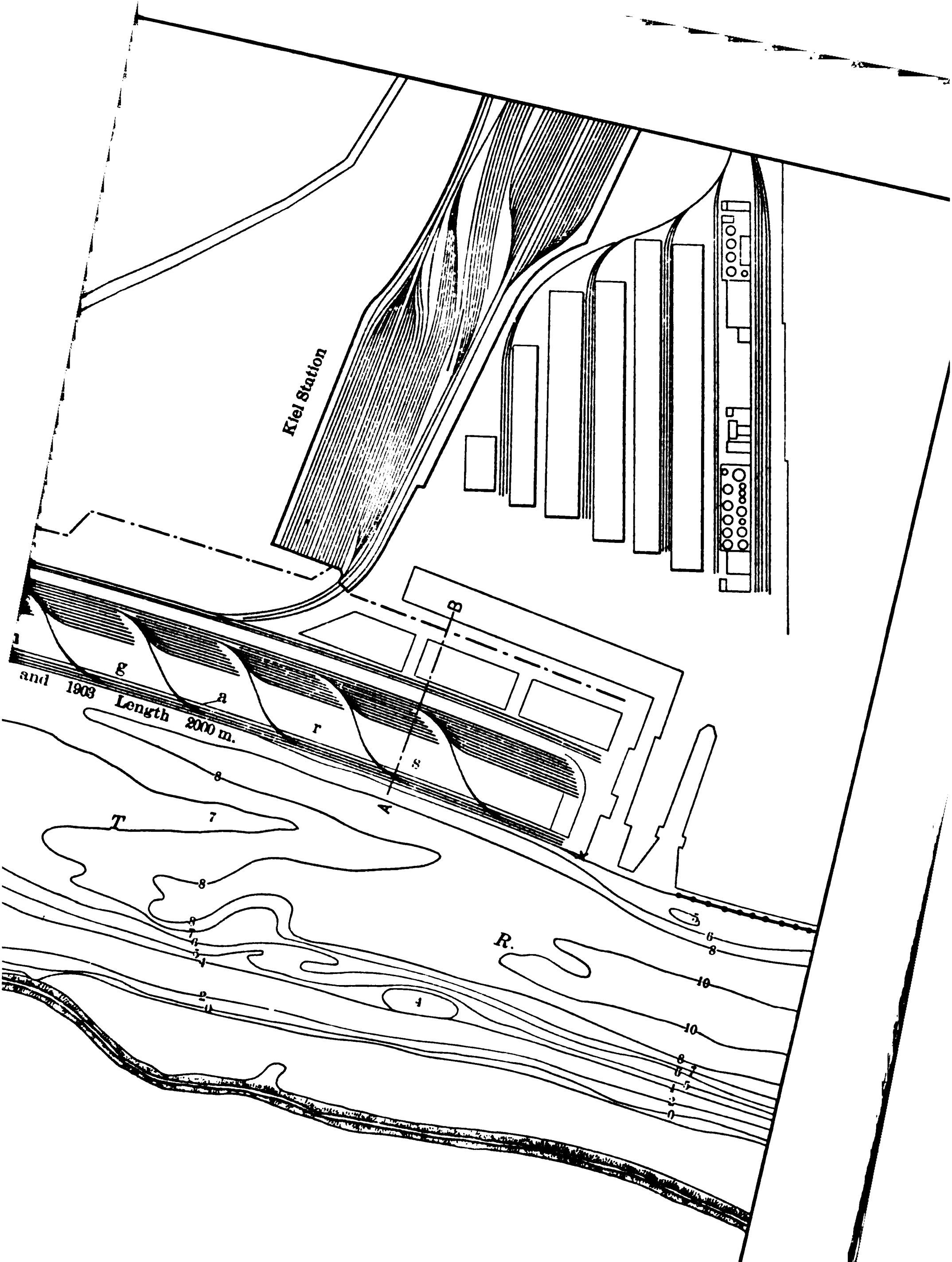


FIG. 421.—ANTWERP



ERMINALS.

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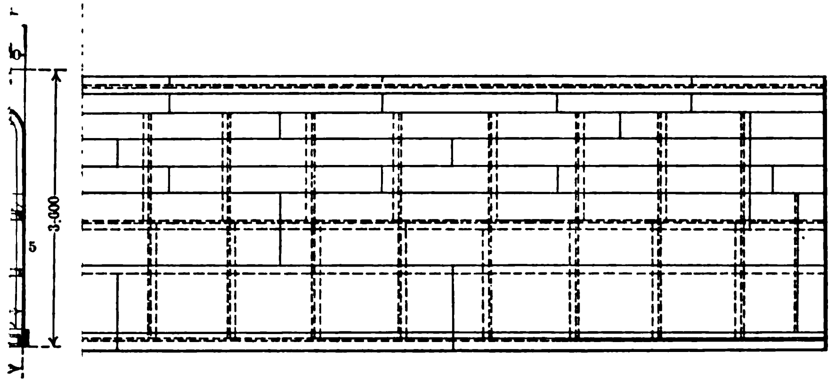
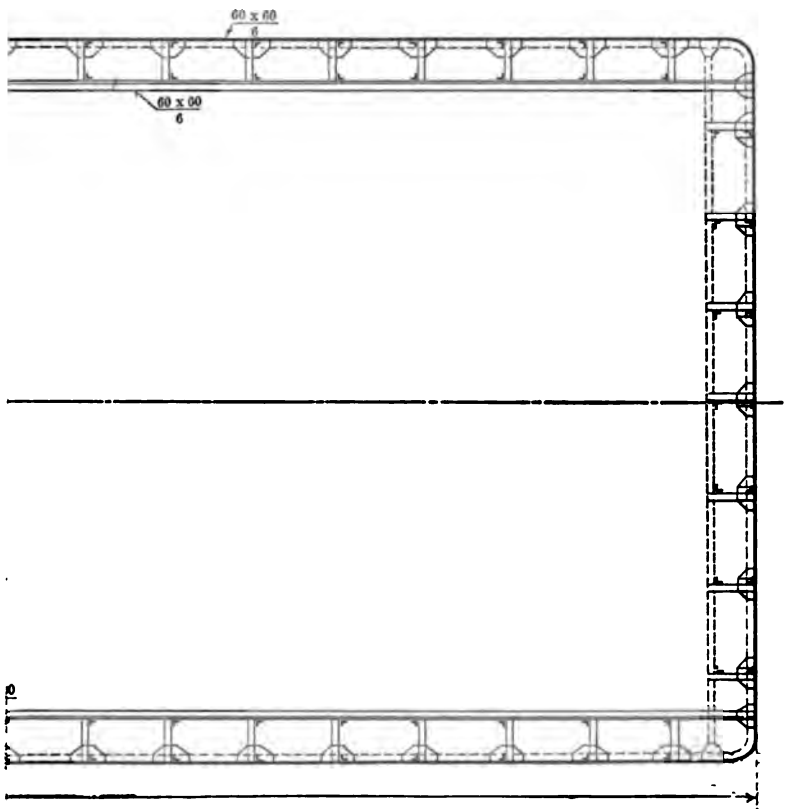
½ ELEVATION



½ PLAN HORIZONTAL E F



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the clay showed that for similar material at the Tower bridge in London 4.4 tons per square foot was allowed; at the Charing Cross bridge 7.2 tons per square foot; at the Canon Street bridge

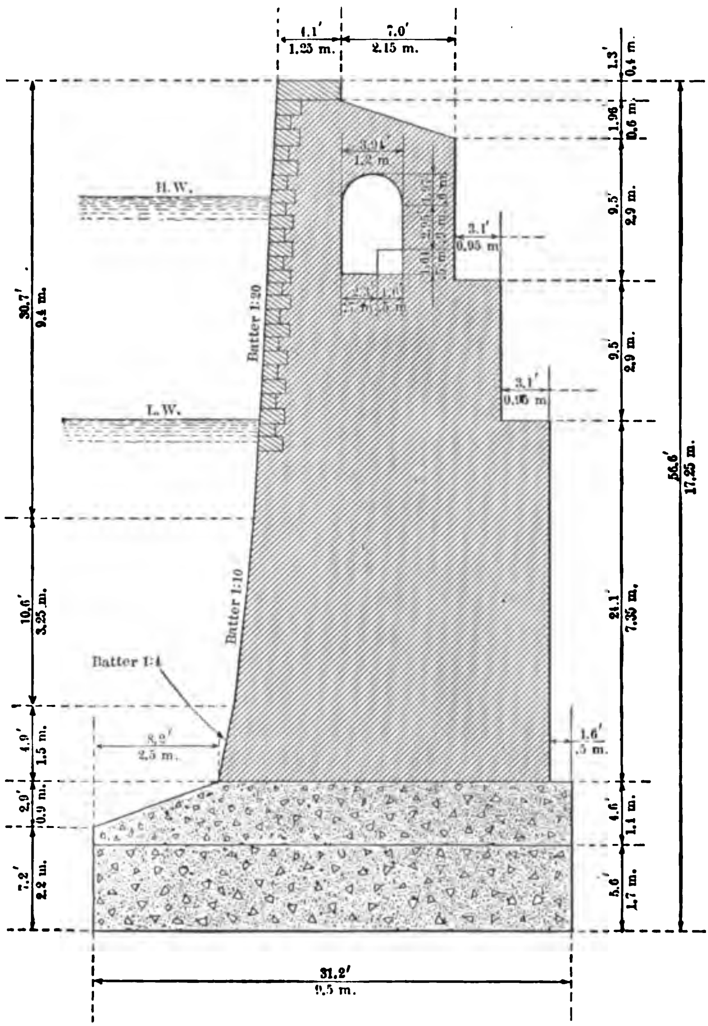


FIG. 422 (*a*).—ANTWERP QUAY WALL SECTION.

4.6 to 5.1 tons per square foot; and at the Kidderpur dock in India 4.8 tons per square foot. The allowable pressure in kilograms per square centimeter finally adopted was 3.3 kg. + 0.22 kg. $\times h$, the factor h being the depth of the foundation below the bed of the stream in

meters. These walls were constructed in sections of 30 meters or about 100 feet in length by means of compressed-air caissons, of which only the steel working chamber was finally left in the completed work. The design of these caissons is as shown in Fig. 422, the working chamber being 5.75 feet high and practically 31.2 feet in width, the caisson proper being generally 11.8 feet in height, varying in accordance with the depth to which the work was to be carried.

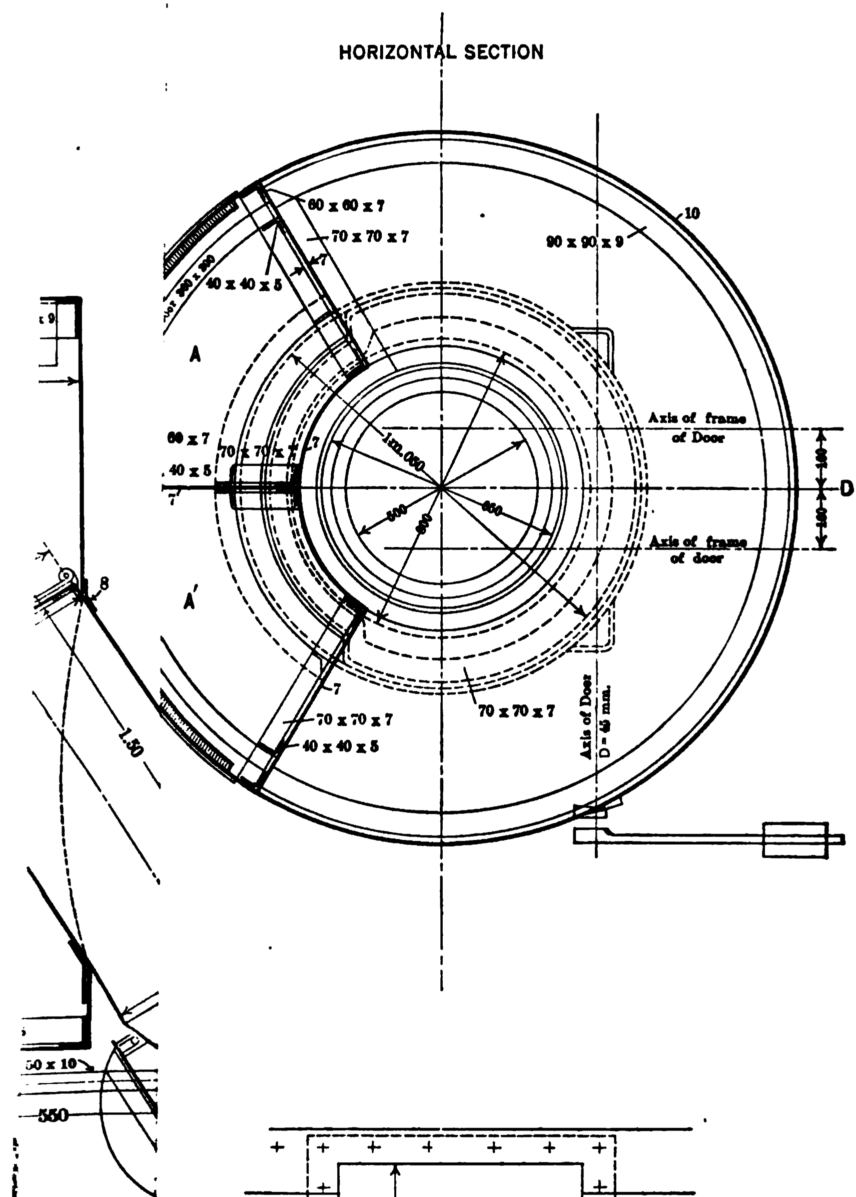
Two steel barges (Fig. 423) were used in the construction and were connected together by a framework spanning the caisson so as to assist in handling the caissons and the movable coffer-dams as shown. These scows and framework carried all of the compressed-air plant, machinery, and material used in the construction. When the steel caissons had been constructed and launched, they were partially filled with concrete as shown in Fig. 424 to land them, and additional concrete added as was found necessary to sink them. The compressed air was then introduced into the chamber, and the workmen entered through the No. 1 and 2 air-locks, and shafts, while the material excavated was removed through the shafts and locks fitted for that purpose. The arrangement of the ingenious machinery in these air-locks is shown in Figs. 425, *a*, *b*, *c*, and *d*.

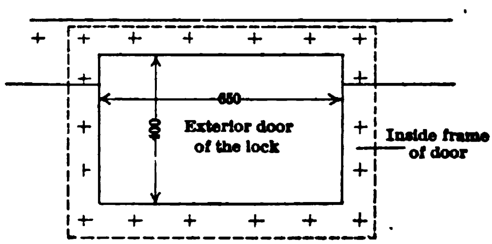
Each caisson was provided with five shafts, the end and center ones being 3.45 feet in diameter and the two intermediate ones of 2.5 feet in diameter. Each shaft was in three equal sections, connected with angle flanges, bolted together with tallowed hemp gaskets. The shell was about $\frac{5}{16}$ inch thick in all of them.

There were four types of air-locks employed as shown in Fig. 425, two of the Pagnard or No. IV type on the intermediate shafts, one No. I on one end shaft, one No. II on the other end, and a No. III or Tchokke lock on the center shaft.

Type No. I, for both workmen, tools and excavation, was 4.6 feet in diameter, 6.6 feet high and of $\frac{5}{16}$-inch metal. It was provided with two inclined tubes for discharge of the excavated material, each tube holding about 7 cubic feet of material. The material was hoisted in buckets on a wire cable running on a differential drum, each bucket holding from about 1 cubic foot to 2 cubic feet each, and being dumped by a man stationed in the air-lock. When the cylinders were filled and the door from the shafts closed, then the doors at the outer end of the cylinders were opened as soon as the air pressure was equalized, and the material dumped. Safety chains prevented the outside door being opened until the inside was closed.

Type No. II was also used by the workmen and for discharging





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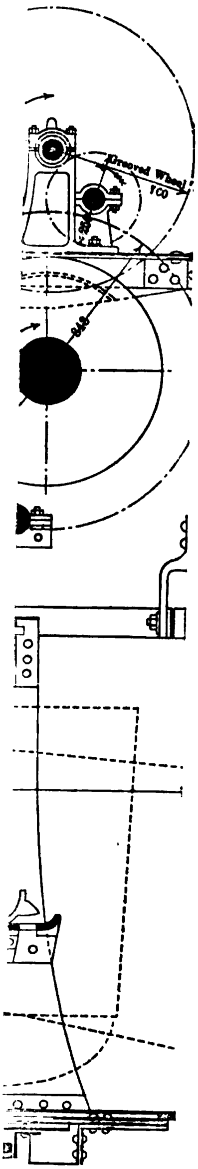
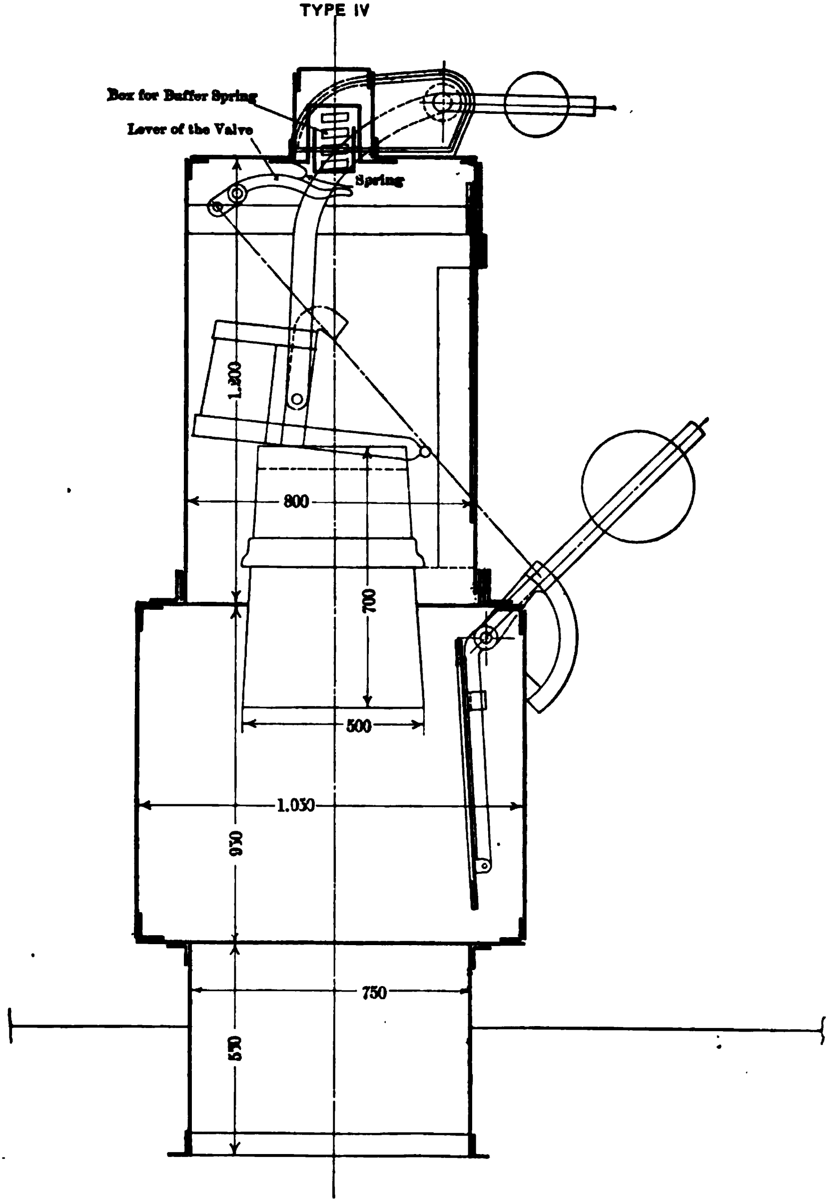

Grooved Wheel

PAGNARD LOCK

TYPE IV



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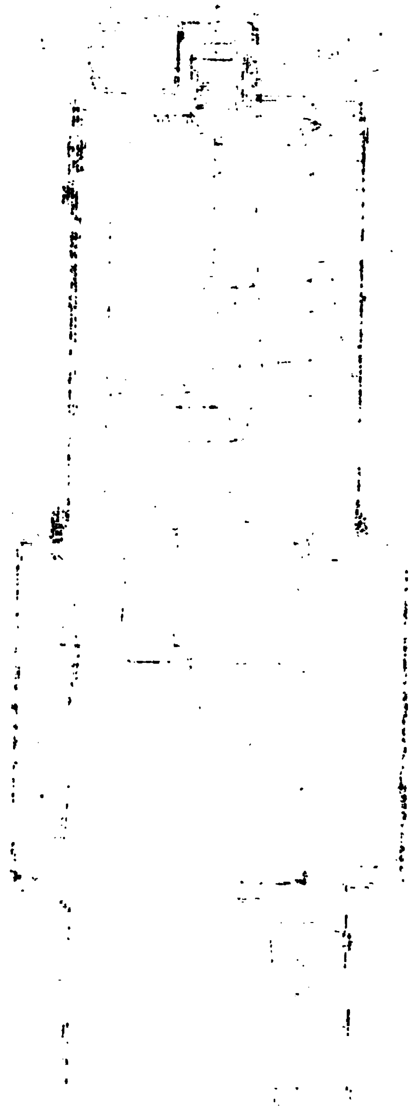


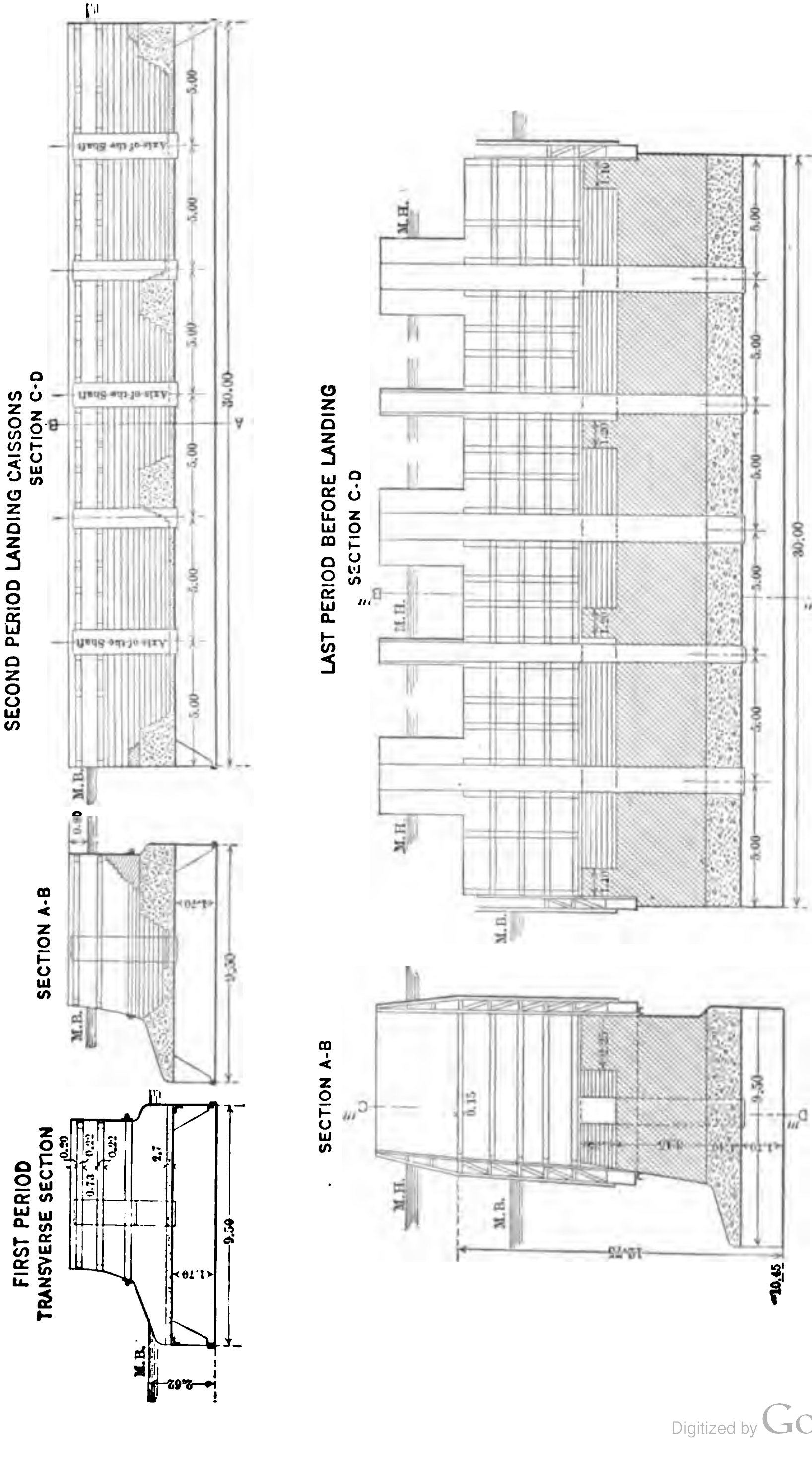


FIG. 424.—LAUNCHING AND LANDING CONDITIONS, ANTWERP CAISSONS.

the excavation. This was 6.2 feet in diameter, 7.67 feet high, with a domed top, of $\frac{11}{32}$-inch metal, and consisted of two contiguous vertical locks, having double swinging doors at the bottom, with vertical sliding doors closing on rubber gaskets and operated by a cable on a drum as shown in the drawing. The buckets used in this lock carried about $2\frac{1}{2}$ cubic feet of the excavated material.

Type No. III, on the central shaft, was a Tchokke lock, especially for the removal of the excavated material, and the passage of shores and tools. The bucket was of 200 liters or about 7 cubic feet capacity

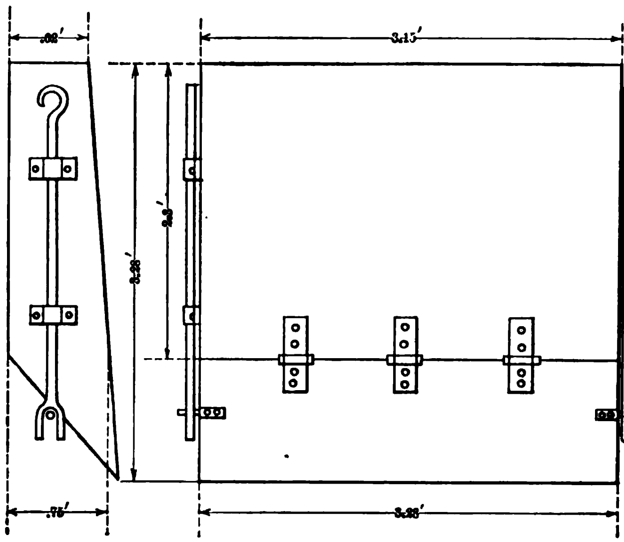


FIG. 428.—NARROW CONCRETE BUCKET.

and in being hoisted it closed the passage between the shaft and the lock, and operated a lever opening a gate and dumping the bucket.

Type No. IV, used on the two intermediate shafts, was called the Pagnard, having been designed by M. Pagnard, superintendent of the work. The bucket used in this lock carried 80 liters or about 3 cubic feet, and on reaching the top operated a lever which opened a valve to the air. As soon as the men outside heard the noise of the air escaping, they closed the door between the lock and the shaft and as soon as atmospheric pressure was reached in the lock, the outside door was opened and the bucket dumped. Owing to the counter-weights employed and the counter-pressure of the air, the maneuvers were all easily carried on.

The method of filling the caisson with concrete (Fig. 426) consisted

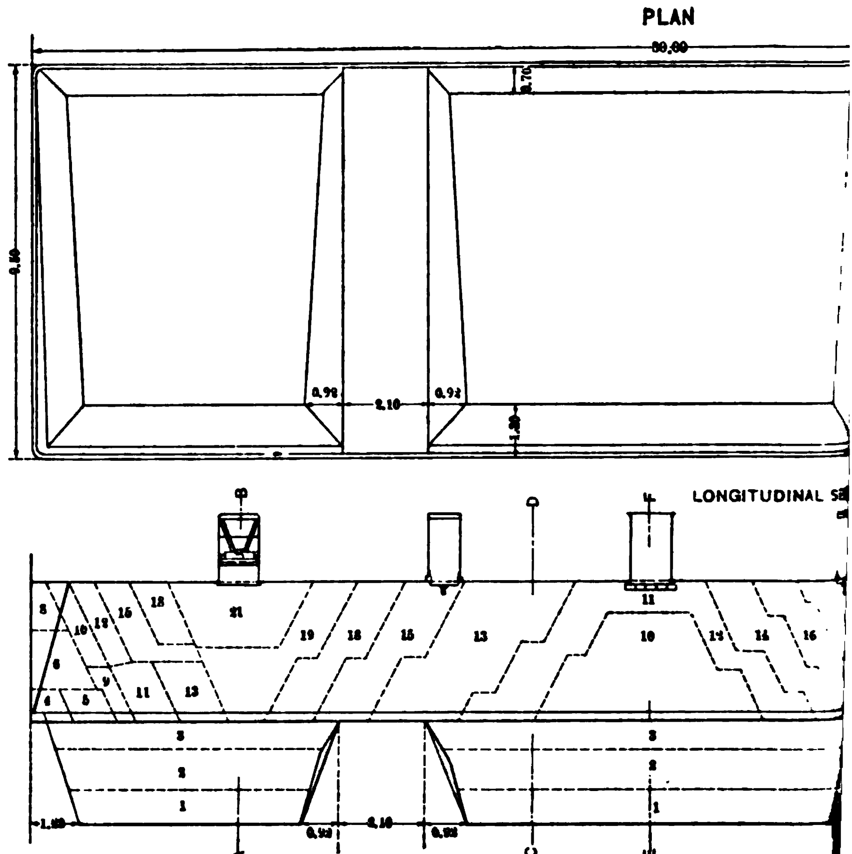
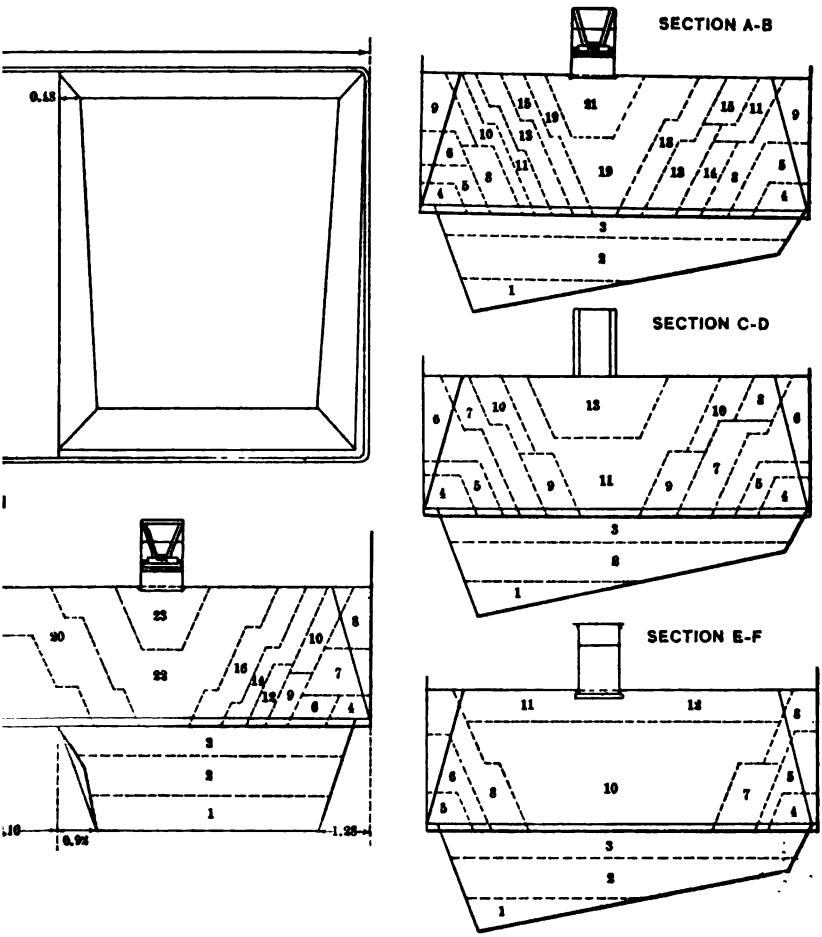


Fig. 426.—Method of (



ING ANTWERP CAISSONS.

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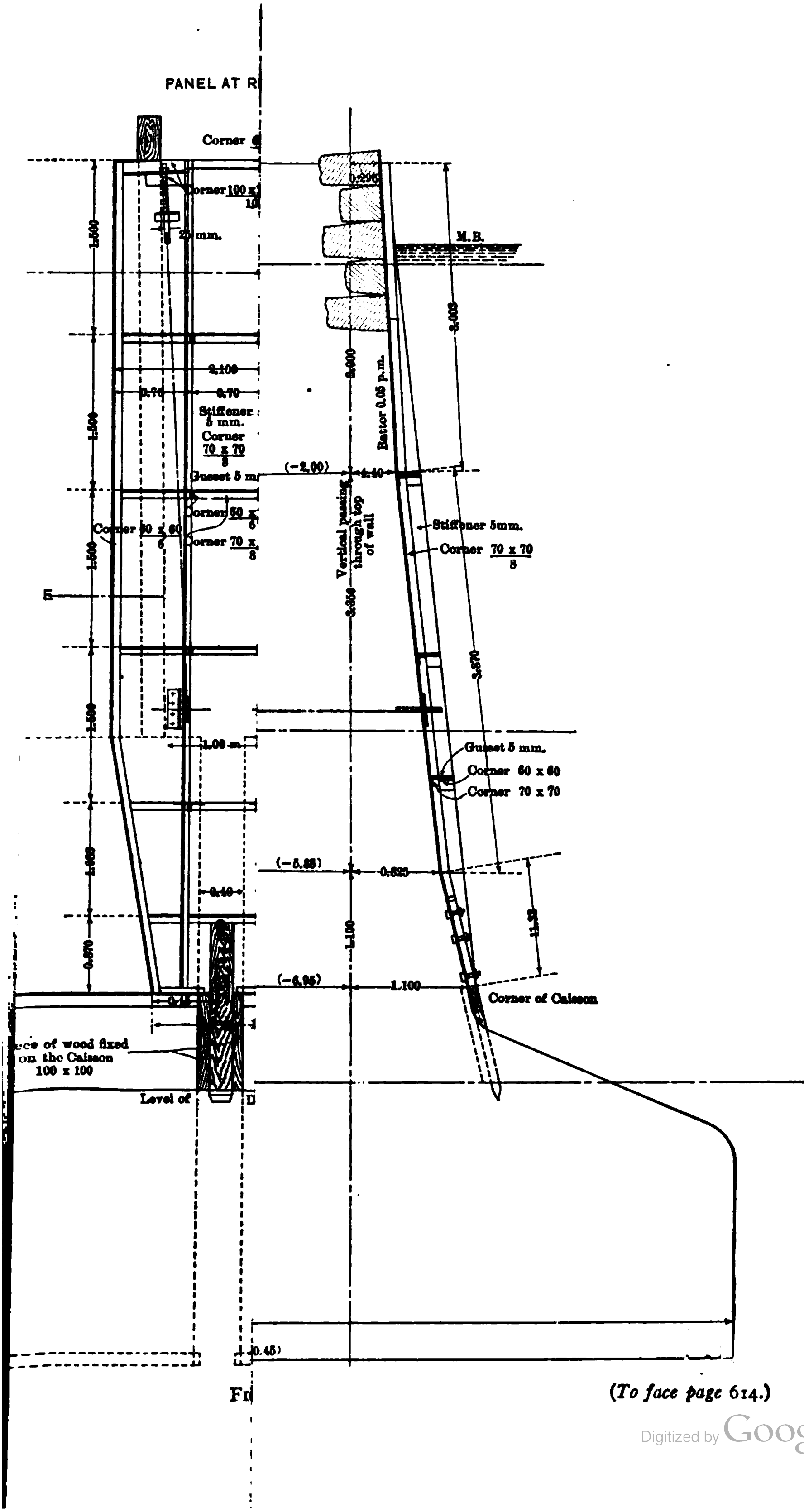

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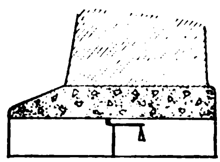
of depositing it through gravity mixing tubes with a charging air-lock connecting with the air-shafts. After two adjoining sections were in place, the intervening space was closed by the movable panels or forms shown in Fig. 427, and the concrete deposited under water with the narrow bottom-dumping bucket (Fig. 428) to fill up the space, which varied from about 16 inches at the bottom to several feet at the top.

During the progress of the work many experiments were conducted to determine the maximum capacity of the foundation. The device



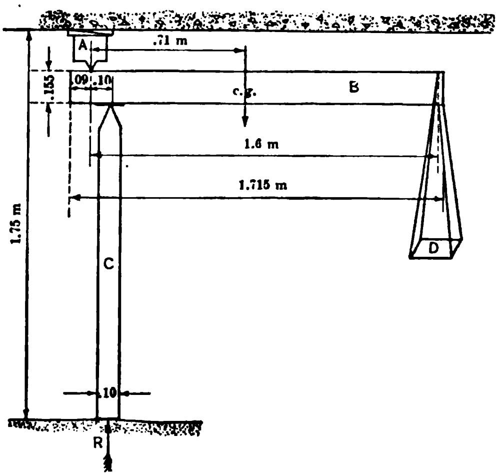


FIG. 429.—CLAY BEARING TESTING APPARATUS, ANTWERP.

employed is shown in Fig. 429, the block *A* being wedged against the roof of the caisson, and this, together with the square timber, had a knife bearing on metal plates on the lever *B*. This lever carried at its outer end a scale platform on which weights were placed to determine the compression on the bottom, the amount of this compression being determined by proportion, after measuring the distance *mn* at the end of the lever as it changed. The resulting maximum load determined by this device amounted to 8.96 tons per square foot, which showed that the amount originally adopted as the safe load

had been a proper one. The unit stresses and data originally adopted for this work are given in the following list:

The constants adopted for construction are given as follows:

Bearing on clay:

$p = 3.3 + 0.22h$,

p = kilograms per square centimeter. [Probably not to exceed 4.5 k. per square c.c.—Author.]

h = depth of foundation bed below bed of stream in meters.

[This is expressed safely for tons per square foot, $p = 3.4 + 0.07h$. But not to exceed 5 T. per square foot. h = depth in feet below bed of stream (see Table L).—Author.]

Maximum compression on masonry	10k.
Coefficient of stability against rotation min	1.8
Coefficient of friction, masonry on masonry	0.7
Coefficient of friction, masonry on clay	0.3
Natural slope of earth fill	35°
Natural slope of clay	60°
Angle of friction of earth fill against inside face of wall	17°
Weight per cubic meter of fill back of wall	1800 km.
Masonry of bricks	1800 km.
Ashlar or cut stone masonry	2300 km.
Beton masonry	2400 km.
Beton (of gravel and brickbats)	2100 km.
Clay full of moisture	2200 km.
Surcharge of solid clay carried by wall per meter	6000 km.

(Multiply weights in kilograms per cubic meter by 1.68 for pounds per cubic yard.)

SCHEDULE OF PRICES, ANTWERP QUAY WALL

Excavation	23,600 cu.yds. @ 14.6c	$3,445.60
Dredging and filling	1,650,000 cu. yds. @ 22c	363,000.00
Wall of quay	6480 lin. ft. @ $246.00	1,594,080.00
Boundary walls		22,500.00
Side channels		22,800.00
Bridges and footbridges		7,500.00
Fenders, bollards, and moorings		14,840.00
Ladders		790.00
Drain to sewer		8,000.00
Aqueducts		16,650.00
Bank protection		34,700.00
Administration, installations, etc		10,800.00
Total		$2,099,105.60
Improvement, total per lineal foot		323.95
Quay wall alone per lineal foot		246.00
Total improvement including all charges, per lineal foot		480.00

ITEMIZED COST, ANTWERP QUAY WALL

Per Running Meter and per Cubic Yard.

Masonry foundation, El. −10.45 to −7.35:			
Caisson, Metal	6050 lbs. @ 0.03		$182.00
Sinking	32.4 cu. yds. @ 1.68		54 50
Beton	37 cu. yds. @ 3.70		136.90
Installations	37 cu. yds. @ 0.23		8.41
			$389.16
Base per cubic yard		$10.50	
Masonry above base, El. −7.35 to +1.00:			
Brick masonry	62.2 cu. yds. @ 3.47.		$215.83
Cut stone	1.3 cu. yds. @ 11.10		14.43
Installations	63.2 cu. yds. @ 0.11		69.52
			$299.78
Middle wall, cubic yard		$4.75	

Masonry above base, El. +1.00 to +6.80:		
Brick masonry	20.45 cu. yds. @ 3.45	$70.55
Ashlar	3.62 cu. yds. @ 10.95	39.69
Cut stone	0.65 cu. yd. @ 19.50	12.68
		$122.92
Top of wall, cubic yard	$4.95	

Above costs would probably be exceeded by 50% in the United States.

DIMENSIONS OF ANTWERP PNEUMATIC WORK

Caissons, regular (64):	
Width	31.2 Ft.
Length	98.4 Ft.
Height	11.8 Ft.
Height working chamber	5.58 Ft.
Plate-thickness	3/16 In.
Brackets spaced longitudinally	3.6 Ft.
Brackets spaced transversely	3.9 Ft.
Weight	73.0 Tons.
Weight per lineal foot	1485.0 Lbs.
Coffer-dam, height	32.8 Ft.
Barges or Floats:	
Width, each	18.0 Ft.
Length	105.0 Ft.
Depth	7.9 Ft.
Distance between the two	27.5 Ft.
Distance deck to truss	25.0 Ft.
Depth of truss	4.9 Ft.
Movable panel height	11.4 Ft.
Sections wall separation	1.32 Ft.
Large Air-locks. Dome Top:	
Diameter	6.2 Ft.
Height	7.7 Ft.
Thickness metal	11/32 In.
Diameter shaft	3.45 Ft.
Thickness metal	5/16 In.
Material Locks:	
Diameter	4.6 Ft.
Height	6.6 Ft.
Thickness metal	5/16 In.
Diameter shaft	2.5 Ft.
Thickness metal	5/16 In.

Angle irons—equivalent size:

40 × 40 × 5 mm. = 1 9/16 × 1 9/16 × 3/16 in.
50 × 50 × 5 mm. = 2 × 2 × 3/16 in.
60 × 60 × 6 mm. = 2 3/8 × 2 3/8 × 1/4 in.
60 × 60 × 7 mm. = 2 3/8 × 2 3/8 × 9/32 in.
70 × 70 × 7 mm. = 2 3/4 × 2 3/4 × 9/32 in.
70 × 70 × 8 mm. = 2 3/4 × 2 3/4 × 5/16 in.
80 × 80 × 8 mm. = 3 1/8 × 3 1/8 × 5/16 in.
110 × 110 × 12 mm. = 4 5/16 × 4 5/16 × 15/32 in.

MILLIMETERS TO INCHES. (NEAREST 32ND)

1 = 1/32	5 = 3/16	9 = 11/32
2 = 3/32	6 = 1/4	10 = 13/32
3 = 1/8	7 = 9/32	11 = 7/16
4 = 5/32	8 = 5/16	12 = 15/32

The considerations that must be taken account of in the design of harbors and particularly in founding breakwaters are the physical and geological data along the shore; the slope of the shore and the distance to deep water; the extreme depth in which work has to be carried on; the force of the waves due to the exposure of the coast, together with the angle of the waves to the coast and the breakwater; and the data of tides, such as their range, direction and force.

The study of the geology will determine the character of the construction to be used, and while occasionally the bottom will be found hard enough to use heavy concrete blocks, either directly on the bottom or after it has been leveled off with concrete in bags as at New Haven, more often mounds of large riprap must be employed to withstand the impact of the waves.

The force of waves as observed by Thomas Stevenson at Skerryvore lighthouse on the Atlantic, in the year 1845, reached 6083 pounds per square foot in the greatest storm, the winter average being 2086 pounds, and the summer average being only 611 pounds. The force observed at the Bell Rock light on the German Ocean was 3013 pounds.

The placing of the material for breakwaters is often carried out by the construction of a timber trestle from which to dump the rock from the cars on which it has been hauled from the quarry, or on which it has been placed from vessels by the aid of derricks; or it may be placed directly by huge Titan cranes from the vessels; or lastly placed or dumped directly from scows. The full discussion of the construction of breakwaters will not be attempted in this work, and the reader is referred to the discussion in *Civil Engineering* by Vernon-Harcourt.

The cost of breakwaters per lineal foot is given for a few instances in the following list:

Marseilles, 30 to 36 ft.	$348 per lin. ft.
Marseilles (new), 30 to 40 ft.	528
Portland, Eng., 48 to 60 ft.	561 (convict labor)
Algiers, 36 to 54 ft.	590 (large beton blocks)
Holyhead 18 to 42 ft.	790
Alderney, 20 feet depth.	822
Plymouth, 40 to 45 ft.	967
Dover, 40 to 50 ft.	1740

The construction of breakwaters with riprap rock has been discussed to a considerable extent in Chapter XXIV, and only a few

examples need be given to illustrate more fully the cross-sections that are used to withstand the terrific force of the waves during great storms. The Holyhead breakwater (Fig. 430) is founded on a riprap or rubble base 400 feet in width which acts as a foundation for the masonry wall surmounting it. The Plymouth breakwater (Fig. 431) has a base 325 feet wide which acts as a foundation for the

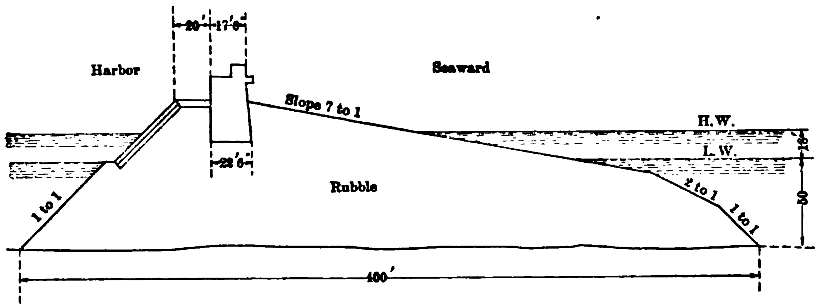


FIG. 430.—HOLYHEAD BREAKWATER.

masonry paving of the crest. Where harder bottom is encountered, concrete blocks of large size are often built into a rough wall forming both the base and superstructure of a breakwater. The breakwater in Osaka Harbor (Fig. 432) is constructed with a rubble or riprap foundation as shown in Fig. 433, which also shows the superstructure wall, constructed of concrete blocks placed with a floating derrick.

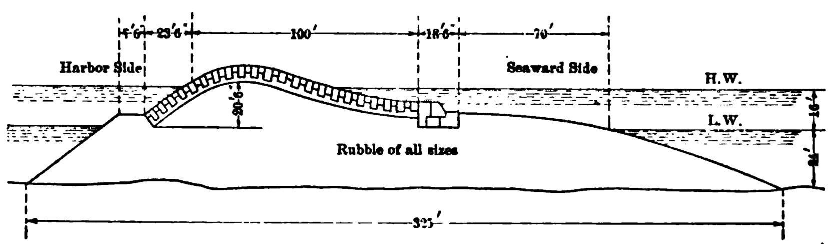


FIG. 431.—PLYMOUTH BREAKWATER.

The method of preparing the foundation and constructing breakwaters is such an important one that the able paper by Henry C. Ripley published in the Transactions of the American Society of Civil Engineers is given as reciting the best methods that can be employed:

Introduction.—Within the last twenty-five years a method of jetty construction has been developed in the United States, which, for

stability and economy, has never before been approached as far as the writer is aware. As no complete description of this method has ever been published, it is the writer's purpose to discuss it in detail, in accordance with its most recent development, and give the reason for each detail, so that those who are unfamiliar with work of this class may appreciate its utility. The order of construction by which bar advance may be minimized, if not wholly prevented, will also be described.

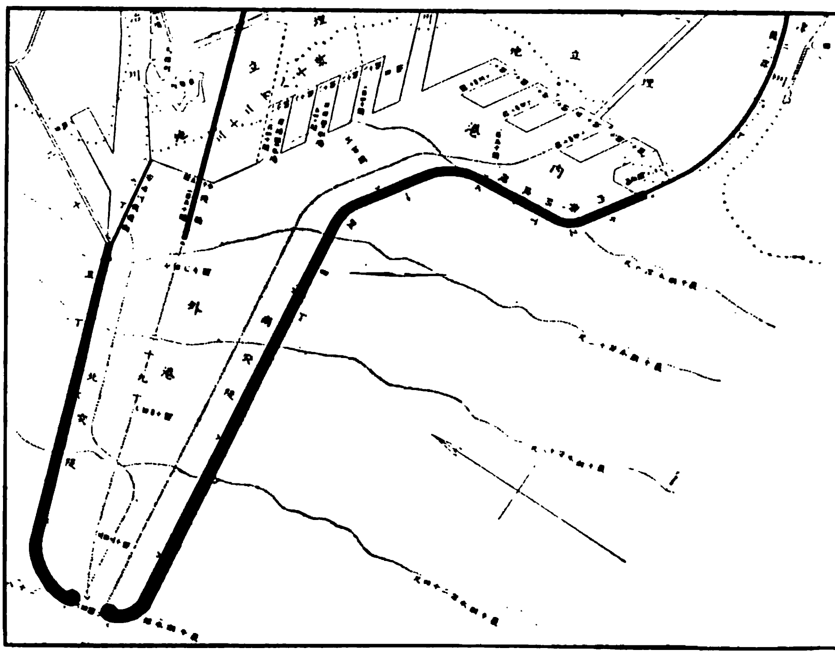


FIG. 432.—OSAKA HARBOR AND BREAKWATER.

Explanation and Definitions.—A jetty consists of the four distinct elements: the foundation, the core, the side-blocks, and the crest or cap blocks. That portion above the foundation, taken together, is the superstructure. The foundation is a thin layer of stone—usually about 3 feet thick—of the full width of the jetty; that part projecting beyond the superstructure is called the apron. The core is the interior portion of the superstructure, and the crest-blocks and side-blocks are the large blocks of stone covering the crest sides of the core. A section of the superstructure will have the form of a trapezoid. The slopes of the sides will be such as the stone assumes

in placing, and will vary according to the degree of exposure to wave action and the size of stone used. Generally, however, they may be kept within the limits of one horizontal to one vertical (1 : 1) and two horizontal to one vertical (2 : 1). The slopes will generally be unequal, because one side will usually have greater exposure to the waves than the other. The top width will vary with the degree of

FIG. 433.—BREAKWATER, OSAKA, JAPAN.

exposure to wave action, but it should not be less than 10, or more than 20 feet. If the latter figure is exceeded, it will indicate either improper construction or unusual exposure to wave action. Naturally the width at the bottom will depend on that at the top, the side slopes and the height of the work. The volume of material, and hence the cost of the completed work, will vary with the top width, the side slopes, and the height; therefore it is important to prevent these dimensions from assuming unnecessary proportions. To

be sure, the height of the jetty will depend on the depth of the water where it is located, and this cannot be controlled. However, by preventing scour in advance of the work, under-scour by cross-currents through or under it, and the undermining of the sides by wave-action or longitudinal scour by current action, much may be done to curtail the height which might be necessary were these details neglected.

Before proceeding further it will be useful, and necessary to a clear understanding, to define the different classes of stone entering into the construction. These are as follows:

Small Riprap.—This consists of small irregular pieces of stone, each weighing from 10 to 100 pounds, which may be handled by one man; it is sometimes called " one-man stone."

Spalls or Chips.—These are pieces of stone smaller than the small riprap and are usually handled with a shovel.

Large Riprap.—This consists of irregular pieces weighing from 100 to 1500 pounds, or even more, of almost any shape which the quarry will produce, although very thin and flat pieces are objectionable because they tend to bread into pieces in placing and it is impossible to make a good bond with them.

Large Blocks.—These consist of more or less regular pieces of stone, weighing from 1 to 10 tons, or more.

Method of Construction.—The foundation must be constructed first. It is made of small riprap, large riprap and spalls. There must be enough large riprap to hold the small riprap in place against wave action, and enough spalls to fill the voids in the riprap and prevent the undermining effects of cross-currents by working through and under the mass. It must also be of sufficient thickness to prevent the breaking waves from jetting through the mass, and washing out the sand from under it. A liberal allowance of spalls will always assure a secure foundation against current and wave action through the mass. Where wave action is very great, it is necessary to place the small riprap first and cover it with the large riprap, in order to hold it in place, the voids in the latter being filled with small riprap. Where wave action is not great, the large riprap may be dispensed with and the thickness reduced.

In fixing the width of the foundation, two things must be considered, namely, the width of the base of the superstructure and the width of the apron. The first will depend on the height and on other conditions which will be discussed later. The width of the apron must be such as to provide for any undermining and settlement due to overfall during or after the completion of the whole work,

and for that due to scour, principally along the channel face. The probable depth of scour can generally be predicted with a considerable degree of certainty, and, by watching the results as the work progresses, the amount of deepening caused by overfall may be closely approximated for ordinary conditions. For those extraordinary conditions which are due to great and unusual storms, the results are more problematical. However, the direction from which great storms come is generally known, and, therefore, the side of the jetty on which the overfall effect will occur. When this happens to be on the channel side, the provision for channel scour will have provided sufficiently for any action due to overfall which is likely to occur. Where there are two jetties, the one suffering from overfall on the outside will be in the lee of the other jetty, and the effect will be thereby diminished. It will generally be found, therefore, that the maximum overfall effect occurs in advance and during the building of the superstructure. When once commenced the construction of the foundation should be pushed as rapidly as practicable, for any delay will permit scour at the unfinished end, which must afterward be filled in with stone, and any deficiency in the full width may permit the edges to drop down, and additional stone will be needed to restore the loss in height. Any temporary deficiency in width which may be necessary in the course of construction, therefore, should be confined to the apron, where a moderate settlement will entail no loss, that is, beyond the base of the superstructure and berm.

Superstructure.—On the completion of the foundation, the superstructure may be commenced, or, in some cases, it may be commenced before the entire completion of the foundation. This will be when its building up will not cause an increase in the cross-currents in advance of the foundation, and where the concentration of the outward flow is not likely to cause advance scour.

The first portion to be constructed is the core, which is composed of large and small riprap and spalls. The small riprap and spalls secure tightness, and the large riprap prevents the small riprap from spreading out and flattening the slope on account of wave action. The flattening of the slope means greater volume, and therefore, increases the cost of the work.

Where the water is deep, or where there is little wave action, the core may be commenced by depositing the stone along the axis of the superstructure, using large and small riprap and spalls in such proportions that the small riprap will fill the voids in the large riprap, and the spalls will fill those in the small riprap. As the core of the wall approaches the elevation of low tide, or, say, within

3 feet of that plane, this form of construction will not suffice to prevent the stone from spreading out and flattening the side slope. When this stage is reached, the sides of the work must be covered with large riprap which must be brought up to a height above the general level of the work in two ridges parallel with the axis of the superstructure. The space between these two ridges will then be filled with small riprap and spalls, and, in some cases, it may be necessary to use also some large riprap, in order to prevent the smaller stones from being washed out by the waves. Then the ridges must be raised again, and the space between them filled as before. This process must be continued until the proper height of the core is reached, the top having been completely covered with large riprap. This height should be such that, with the addition of the crest-blocks, the work will be brought to the full height required for the superstructure after consolidation has taken place.

The allowance for consolidation is largely a matter of judgment. It will depend on the height of the wall, the character of the stone used, the intensity of wave action during construction, the number of unfilled voids in the mass, and the pounding it is likely to receive from storm waves subsequent to completion. The pounding of waves on a mass of stone sets up a vibration which causes the points of contact of the different pieces to rub on each other, and this, by wearing away the stone, permits the mass to consolidate. When this process has been continued for a certain length of time, however, this vibration becomes ineffectual in reducing the bulk, the structure practically ceases to shrink, and the mass becomes stable. Moderate wave action on the work during construction is not an unmixed evil, as it helps the stones to pack together and consolidate, and thus the subsequent shrinkage is reduced. The more nearly water-tight the jetty is made, the more efficient it will be because there will be less leakage and waste of the water which its purpose is to constrain, and because of the smaller quantity of sand which can pass through it to the injury of the channel.

Having regard for these considerations, the allowance for consolidation will vary from 6 inches to 2 feet; but, where the work is subjected to the usual wave action during construction, the shrinkage will rarely exceed 1 foot.

As soon and as fast as the core is finished, the side and crest-blocks should be placed, for otherwise a severe storm would damage the uncompleted work. The side-blocks should be placed one at a time, and allowed to roll down until the sides are completely covered from the bottom up to about the elevation of low tide. Above this

elevation each block must be carefully placed so as to fit as closely together as practicable. As soon as the top is reached, the surface of the core must be leveled up with small riprap, and, if there has been any loss in height by the washing down of the crown, the proper elevation must be restored before capping. The cap-blocks will then be selected, using only those pieces which will give the proper height to the crown, and these should be placed so as to leave as little of the sides as practicable exposed to the waves. The side-blocks should be disposed in relation to the cap-blocks so that the waves will glide over the latter rather than strike with full force against their sides. When these blocks are properly placed, the surface of the core is completely covered, and when the inevitable shrinkage due to consolidation takes place, these blocks wedge themselves together in such a way that the heaviest storm waves cannot dislodge them.

Where one side of the structure has greater exposure to wave action than the other, the larger blocks will be placed on that side, and, as the greatest force of the waves is exerted on that portion of the work above the plane of low tide, much care must be taken in building this portion. This is greatly facilitated, however, by the fact that it is always exposed to view at low water, and there can be no uncertainty as to the position of each and every block.

Stone.—The character of the stone to be used in a work of this nature is of considerable importance. Hardness and weight are prime requisites, especially for the side and crest-blocks, and, for these, granite, gneiss, limestone, sandstone, or other stone of considerable specific gravity, should be used. The value of a stone to resist wave action varies directly with its weight under water and inversely with its surface exposure. Its value depends on its specific gravity, and to an extent which is not always fully realized. Suppose two stones, *A* and *B*, each having the same weight (say, 1 metric ton or 1000 kg. in air), *A* having a specific gravity of 2.1, and *B* a specific gravity of 2.7. Then *A* will have a weight, under salt water, of 496 kg., and will displace 504 kg. of water, while *B* will have a weight of 608 kg., under the same conditions, and will displace 392 kg. of water. Then *B* will have a weight under water $22\frac{1}{2}$ per cent. greater than *A*, a volume of 22 per cent. less, and a surface exposure to wave action $15\frac{1}{2}$ per cent. less than *A*. Therefore, it will have 38 per cent. more stability or power to resist wave action than *A*, while its weight in air is exactly the same.

For the foundation and core there is not the same economy in the use of heavy stone, but the large riprap (the main purpose of which

is to resist wave action), if of high specific gravity, will enable the core to be built with a steeper side-slope, and, therefore, will conduce to economy in the quantity of stone required for a given work. For this reason a limestone is generally preferred to a sandstone of the same grade as to hardness, because of its greater specific gravity. For a foundation in deep water and for the interior of a core, a stone of less hardness is not altogether objectionable, because the chips which result from handling help to reduce voids and consolidate the work sooner. In some cases, the use of a stone of small specific gravity for the foundation will give satisfactory results, with some economy in tonnage.

Order of Construction.—When jetties are designed to control the flow of water across a sand-bar, the order of construction is of vital importance. The order in which the material should be put into the work has already been given under "Method of Construction." It is here proposed to consider whether the work should be built from the shore outward or from the outer end shoreward.

Suppose that two parallel jetties are to be constructed at the mouth of a river, or at the entrance of a tidal harbor, to extend from the shore out across the bar; and suppose that the foundation of each has already been constructed. The position of the bar is determined by the equilibrium of the forces, one set of which tends to push it seaward, while another tends to push it back toward the gorge. When these forces are equal, the distance of the bar from the gorge is constant.

If the superstructure is commenced at the shore ends and extended toward the outer ends, it will have the effect of advancing the gorge toward the bar, with the resulting advance of the latter. If the work progresses rapidly, the jetties may overtake the bar advance and get across it, but such a result will always be at the cost of a considerable extension of the jetties beyond that originally required. If the work of construction progresses slowly, the jetties may never reach and cross the bar, and the expenditure may become so great that the project may be abandoned without accomplishing the purpose for which it was designed.

If, however, the construction of the superstructure be commenced at the outer end of the jetties and continued shoreward, there can be no gradual advance of the gorge toward the bar with the consequent bar advance. On the contrary, as the completed work is extended, the waterway will suffer a contraction, the old gorge will disappear, and a new one will be established at the outer end of the jetties at a point beyond the crest of the bar. With the

increasing contraction of the waterway, there will be an increased current, both between the jetties and laterally through the gaps between the completed work and the shore on either side of the channel; but, as the foundation work of the jetties will prevent enlargement laterally, deepening of the channelway between the jetties will be inaugurated, and, as the greatest tendency to scour will be at the outer end of the jetties, deepening will commence at this point and extend backward with the advance of the completed work shoreward. The material thus eroded from the channelway will be carried beyond the outer ends of the jetties, where it will be either swept to one side by the littoral current or deposited in the deep water farther out.

Another advantage in working from the outer end toward shore is the facility afforded in construction. The work will thus be carried on in the lee of the finished structure, and in this way the number of possible working days is considerably increased. Where the work is being done with a floating plant, this increase in available working days may amount to from 50 to 100 per cent., with a corresponding saving in operation expense. Even where the work is being done by using a trestle, the construction is greatly facilitated by being in the lee of the finished work, and much time may be utilized which otherwise would be lost.

For a single curved jetty, a detached breakwater, or a training wall, where waves and current are encountered, the same general method and order of construction should be followed.

Where the conditions are favorable for the construction of a trestle along the site of the work and the cars of stone are run directly on it, a most convenient method of construction is furnished. It enables the work to be carried on during weather which might be so rough that it would prevent the use of a floating equipment. It also enhances to a considerable extent the rate of progress. The floating equipment, however, is very convenient for placing the foundation, and, on a large work a combination of the two systems may prove advantageous. It is beyond the province of this paper, however, to enter into the subject of the method of handling the stone. Either method will give entirely satisfactory results.

CHAPTER XXX

FOUNDATIONS FOR DOCKS AND LOCKS

THE building of the foundations for drydocks and locks has probably caused more trouble and financial loss than any other kind of foundation work. In the case of drydocks this has been due first, to the very great loads that have to be supported; second, to the generally inadequate borings; and third, to the very insufficient coffer-dams that are usually constructed around the entrance to the docks.

The first difficulty due to carrying the enormous loads can be taken care of by careful engineering design, provided the exact character of the bottom has been predetermined. Where the bottom is soft, piling, of course, must be driven and properly spaced to carry the loads of the dock and vessel with at least a factor of safety of six. The distribution of the load over the piling is nearly always effected by an exceedingly thick base of concrete. The spacing of the piles and the thickness of the bottom are shown in Fig. 434 for the Government Drydock at League Island, Pa., a discussion of which will be given under the description of that work. Where piling are not employed, the load must be distributed by a thick bed of concrete, or else steel reinforcing used to properly strengthen the bottom concrete where there is any considerable likelihood of settlement.

The second difficulty may be largely obviated by making core borings as described in Chapter XXII, instead of depending on wash borings, or some other unreliable method.

The third difficulty, due to a poor coffer-dam, may be obviated in one of two ways. The coffer-dam, if used, should be as carefully studied out and as carefully designed as any portion of the permanent work, or else if it seems probable that a coffer-dam, by reason of the great head and the poor material in which it is to be built will be a failure, the dock should be constructed without a coffer-dam, as was in one case proposed by the author, and also used at Kobe, Japan, a description of which is given in this chapter.

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The author's connection in a consulting capacity with the construction of three large docks, and an intimate knowledge of the construction of others has served to emphasize the foregoing matters very forcibly. One of the docks was enclosed by a very simple but very well-designed coffer-dam, and due largely to the fact that the character of the bottom was well known, very little trouble resulted, except that by additional expense on the coffer-dam the amount of pumping could have been very materially reduced.

In another case the bottom was soft mud, and the coffer-dam was constructed of two rows of sheet and round piles braced, and firmly tied together. The space between was filled with fairly good puddle and the material banked up on both sides. The site for the dock had been first excavated with a suction dredge, and when the attempt was made to pump out the excavation the coffer-dam gave way, due mainly to the bottom being too soft to hold the piling. Had a series of cribs been sunk and filled with puddle, sheet-piling driven on both sides of the cribs, and the puddle material banked up on each side, a coffer-dam would, in all probability, have been successful. After the failure of the coffer-dam the author recommended one of two things: first, to construct a coffer-dam at the middle of the dock so as to construct it in two sections, building the outside coffer-dam while the first section of the dock was being built; or else to abandon the coffer-dam idea entirely, to drive all the foundation piling with followers, to cut them off under water (if necessary), and deposit the entire concrete for the dock by trêmies in forms set by divers without any pumping being done until the dock itself could be pumped out and be properly finished on the inside. The work done at Kobe by this method was described just at this time, and was a strong argument in favor of this method. Two plans were finally submitted for consideration—one being a sheet-pile box around the entire dock, braced with an elaborate system of timber struts, and the other one the method just described for building the dock under water. Owing to a change of contractors, the sheet-pile method was adopted, but undoubtedly proved to be the more expensive on account of the large amount of pumping required. The principal reason for one of these methods being required was that along one side of the dock site a seawall had already been built on a pile and timber platform foundation, and an enormous leakage under this wall made it impossible to cut off the water by using only a crib coffer-dam around the entrance.

A third case of trouble in building a dock was where a coffer-dam improperly designed had been constructed, and a great deal of

trouble occurred that might have been obviated by a properly constructed one.

A fourth instance with which the author had intimate contact had a very simple coffer-dam constructed of guide piles, wales, two rows of tongue and groove sheet-piling, with a poor class of puddle placed between, and a bank of sand and gravel on either side. The only reason why this coffer-dam was successful was that it was built at a high elevation on the original bottom, and in a rough semicircle quite a distance in front of the dock entrance so that when the site was pumped out and the excavation made with steam shovels, the excavation did not approach closely to the coffer-dam, and no very great head had to be sustained by the coffer-dam. Had the excavation been made by a dredge to the full depth of the excavation at the entrance to the dock, a similar coffer-dam would undoubtedly have been a failure, with such an increased head.

A fifth example, where the bottom was of a very porous coral formation, trouble could undoubtedly have been avoided by adopting the under-water method without a coffer-dam, at least for the bottom concrete, including also the base of the side walls. Considerable trouble was experienced by the pressure of the water on the bottom through the porous coral breaking up the concrete, and had the under-water method been adopted, it would undoubtedly have been necessary to reinforce all concrete with mats of reinforcing bars lowered to place and properly located by divers.

The United States Government Dry Dock at League Island, Pa., (Fig. 434) is 797 feet 6 inches in length, and 102 feet $7\frac{1}{4}$ inches wide at the top. The piling was spaced transversely as shown in the section, namely, 3 feet 4 inches center to center under the floor of the dock, and 2 feet 6 inches center to center under side walls, with the longitudinal spacing being uniformly 3 feet 4 inches. The thickness of the concrete over the top of the piling in the original design was 9 feet, but this was changed to 10 feet for the final construction. To prevent any trouble from the up-lift of the water underneath the concrete, the piles underneath the floor of the dock were to be notched as shown in Fig. 434 (*a*). In the author's opinion the piling should have been spaced considerably closer for nine or eleven rows longitudinally through the center for supporting the keel blocks, so as not to depend too much upon the distribution of the load by the concrete. Considerable trouble has been experienced on many docks by cracks appearing between the bottom and the wall concrete. This has been taken care of in a number of designs by the closer spacing of the piles under the walls, and by the

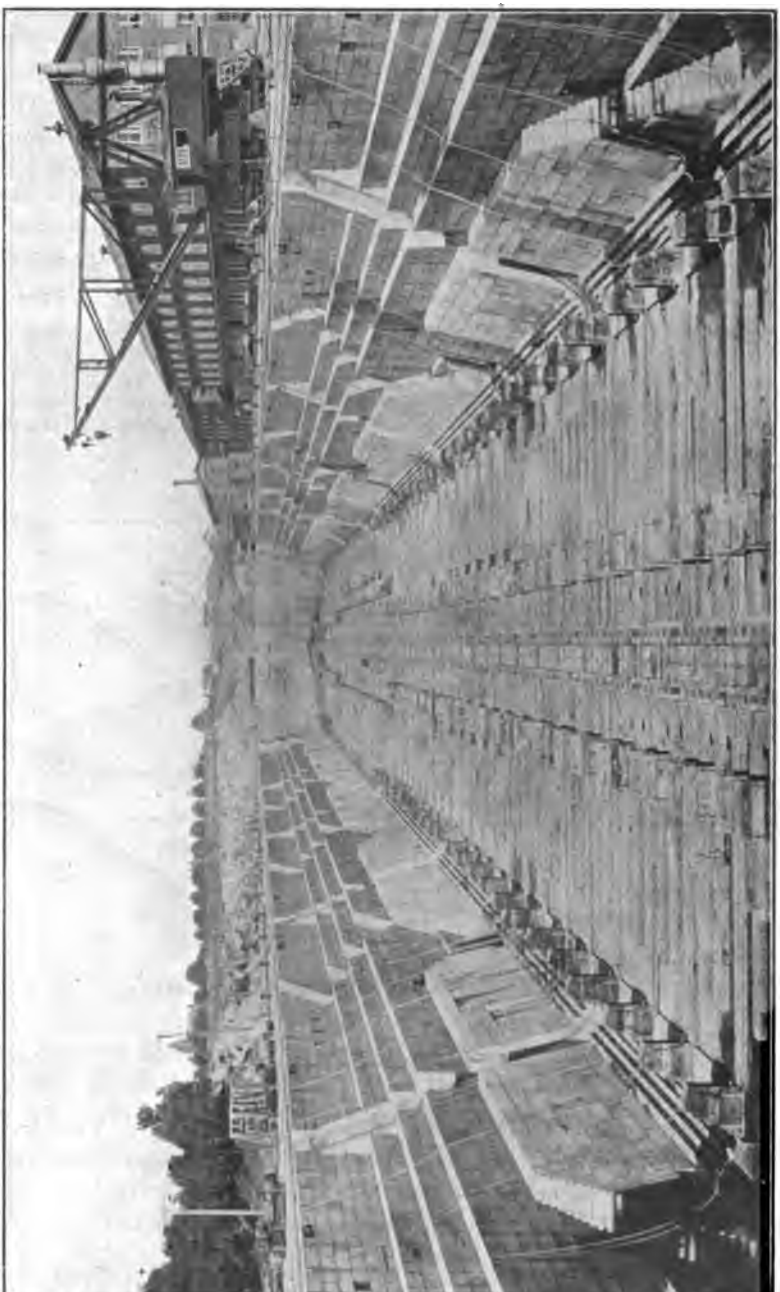

FIG. 435.—PUGET SOUND NAVAL DRY DOCK.

use of steel reinforcing connecting the bottom and the wall concrete, within such surfaces as were liable to undergo tension stresses.

The foundation for the dry dock at the Puget Sound Navy Yard was a hard cemented gravel, so that no piling were necessary. This dock was built of concrete with a considerable amount of steel reinforcing, and the interior of the walls (Fig. 435) or altars were lined with dressed granite.

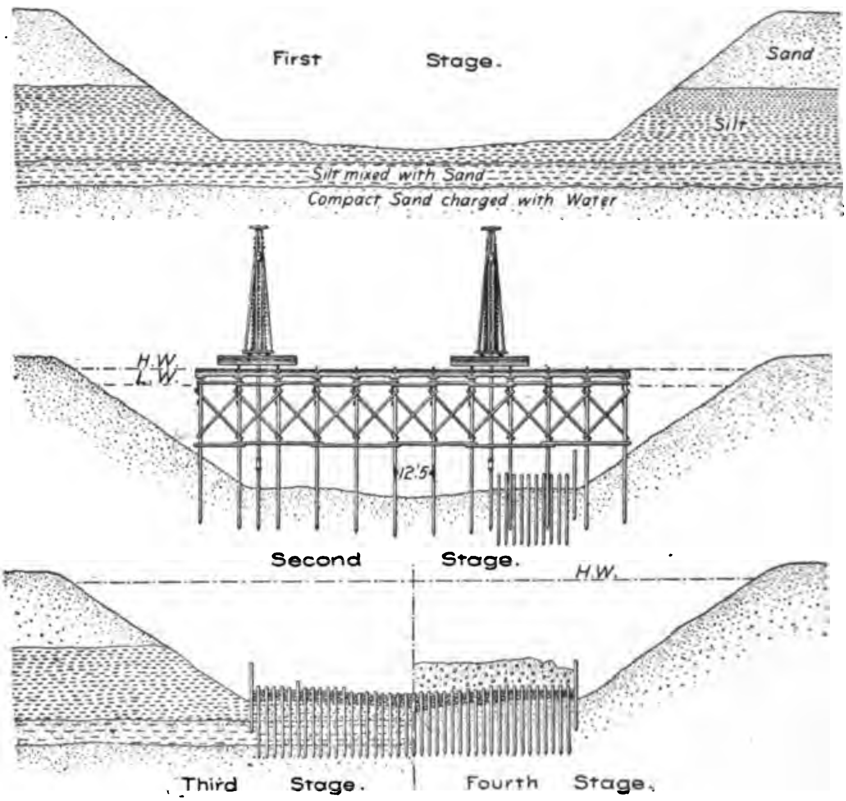

FIG. 436.—KOBE DRY DOCK CONSTRUCTION.

The description of the Kobe dock by Dr. Genjiro Yamasaki has been quoted in full from the *Engineering News* of September 24th 1903. The different stages of the work are shown in Figs. 436 and 437, and a plan and longitudinal section of the completed dock are shown in Fig. 438.

" The Kawasaki Dockyard is situated near the mouth of the old Minatogawa in Kobe, which is at present the greatest trading port

of Japan. While the dockyard was owned by Government, the need of a dry dock had already been felt, and several efforts had been made to select a proper site for one, but owing to the bad nature of ground along the general coast line of Kobe, the task of building such a structure was given up as an impossible achievement. The result was the construction of a patent slip in the dockyard which accommodated vessels up to 2000 tons and which is still in good working order. A few years after, viz., in 1886, this dockyard was

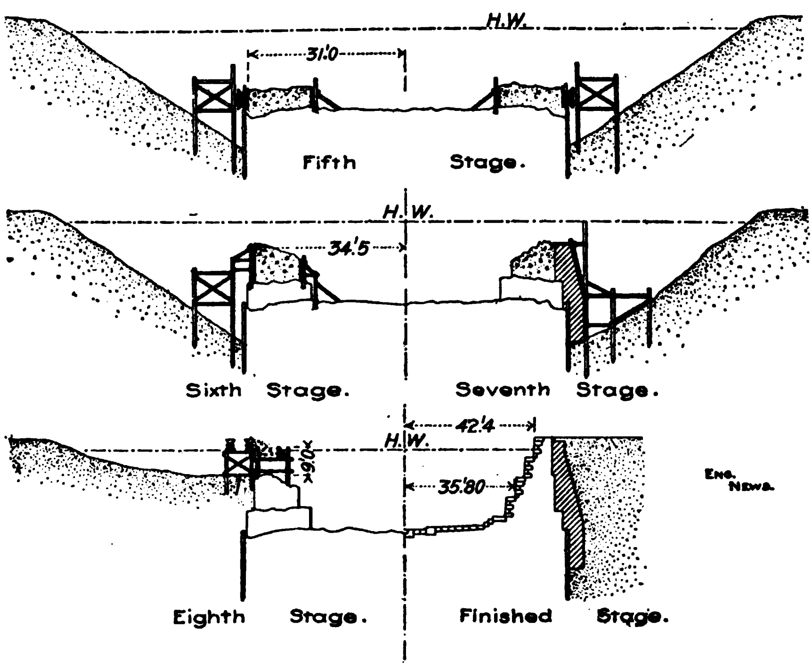

FIG. 436*a*.—KOBE DRY DOCK CONSTRUCTION.

given over to Mr. Shozo Kawasaki, who still maintains an active interest in its welfare.

"Owing to the sudden increase of trade in Kobe from about the year 1893 and to the consequent increase of large vessels frequenting the port, a dry dock became an urgent necessity to meet the requirements of these vessels. Investigation into the subject was, therefore, again taken up, and after a careful study of the nature of the ground and the methods of dealing with it, it was finally decided to start the work. Just at this time, October, 1896, the dockyard, which had been Mr. Kawasaki's property for about ten years, was transferred

to a joint stock company, which is the present Kawasaki Dockyard Co., Limited. The president of the company is Mr. Kojiro Matsukata, son of the ex-Premier of Japan, and the vice-president is Mr. Yoshitaro Kawasaki, son of Mr. Shozo Kawasaki.

"The work was begun in November, 1896, and the first vessel was docked in June, 1902, thus taking nearly five and one-half years for its completion. The general dimensions of the dock are given in the preceding table.

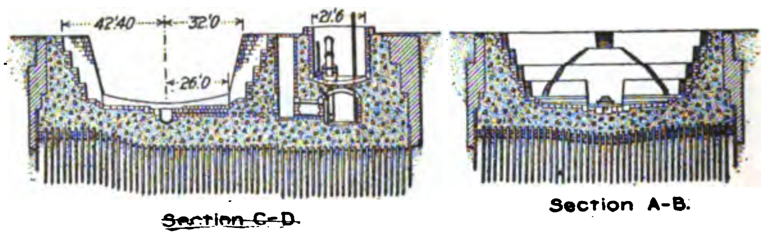

FIG. 437.—KOBE DRY DOCK CROSS-SECTIONS.

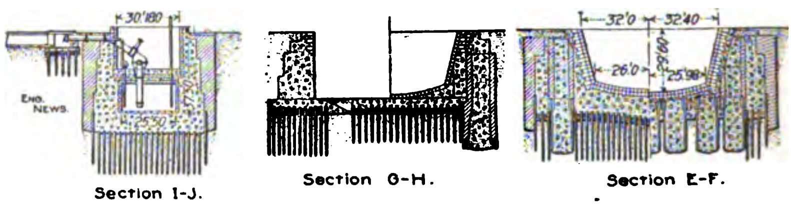

FIG. 437a.—KOBE DRY DOCK CROSS-SECTIONS.

"The dock accommodates vessels up to 5000 tons; its capacity is equal to 20,760 tons at high water.

	Shaku.*
Extreme length, outer caisson stop to toe of wall at head	428.0
Length on the floor	392.0
Width of body at coping level (narrowest part)	79.8
Width of entrance at coping level	64.0
Width of entrance at bottom	52.0
Depth of sill below coping	28.0
Depth of sill below high-water spring tide	24.0
Range of spring tide	5.5

* The shaku is almost equal to the English foot, it being equal to 0.9942-foot; in this article "foot" is to be understood as a "shaku."

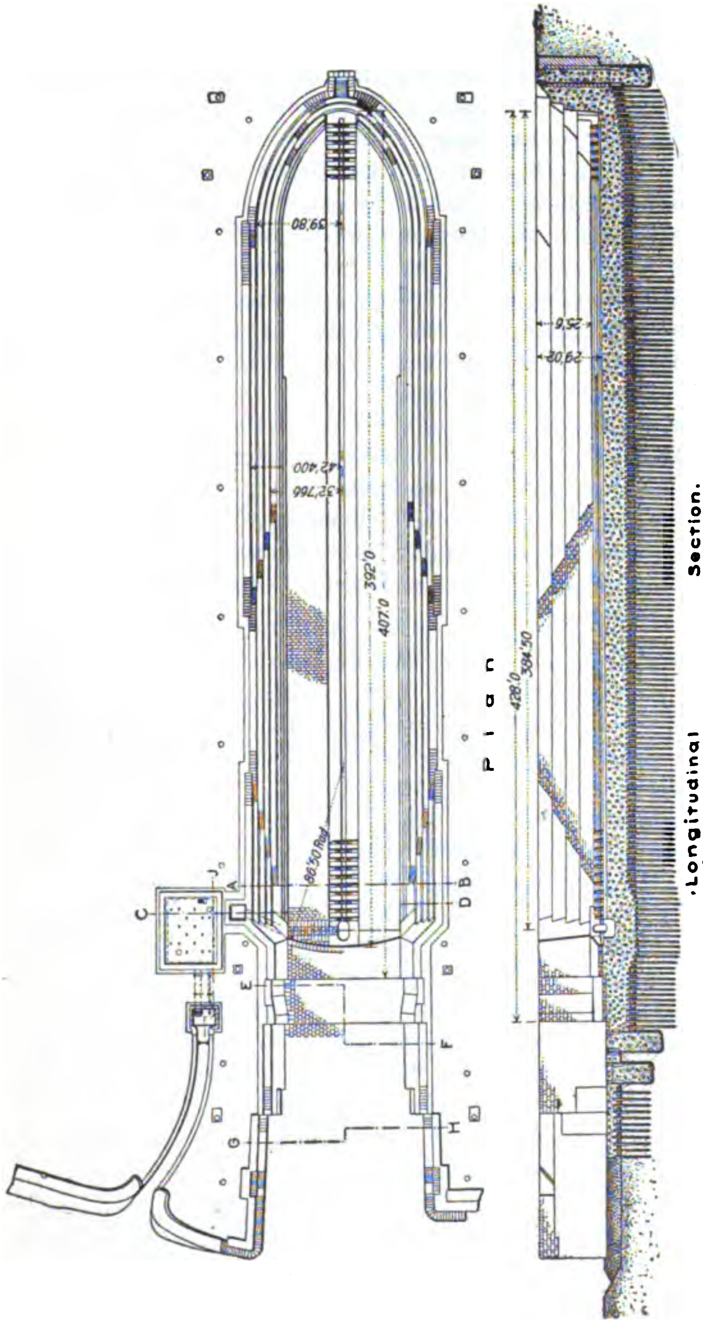

FIG. 438.—DRY DOCK AT KOBE, JAPAN.

"*Site.*—The first idea was to build the dock in the northern corner of the yard, but after considering the arrangement of workshops, building slips, etc., it was decided to select the southeastern corner for its site. The area of the site thus chosen being too small for the dock, necessary space had to be obtained by reclaiming the foreshore. As the easterly wind is the one to be most feared, the direction of the center line of the dock was turned as far north as possible, and that of the finished dock is north 46° 50′ east.

"*Geological Nature of Ground.*—Two borings on land, two borings on the foreshore, and one test pit near the coast were driven to ascertain the geological nature of the ground. Altogether there is a slight inclination of the strata towards the sea, their general arrangements are:

From H. W. spring tide.
(o) to −20 ft..................Sand.
From −20 to −45 ft...........Silt.
From −45 to −51 ft...........Silt mixed with sand.
From −51 to −52 ft...........Broken granite.
From −52 to −73 ft...........Compact sand charged with water.

"This −73 feet was the greatest depth ascertained by the borings as the lower strata were fairly well known from the experience which Japanese artesian well borers had obtained while driving wells in the vicinity of the yard. According to their information, this sand stratum extends as far down as −90 feet; then follows another layer of silt about 33 feet in depth to −123 feet below which there is another layer of compact sand. The depth of this sand stratum is not known, but it is certain that it extends as far as −168 feet, the lowest limit ever reached in the vicinity of the site.

"The silt layer, which lies below the uppermost sand stratum on land, forms the sea bottom on the sea part. This silt bottom was so soft that, while the boring was being done in front of the shore subsequently reclaimed, a boring rod, accidentally dropped, sank about 12 feet by its own weight, and later, while constructing the coffer-dam in this part, great trouble was experienced owing to the sliding in of the trench made for the puddle.

"The test pit, sunk near the seashore, was 5 feet in diameter and its wall was made of wooden planks strengthened inside and outside with angle irons and iron bands. When its lower extremity reached −40 feet, or it was sunk about 18 feet into the silt, the inside of the pit was dried up, and a wooden pile was driven. When the lower end of this pile was down to −53 feet, water, which found its way along the pile, appeared, and it rose so fast in the pit that it was

filled with water from −26 feet to within 9 feet of its top edge (+1 ft.), or 18 feet in fifty minutes, or at the rate of about 12 tons per hour. This water, which exists in the stratum of the compact sand, has a sufficient head to raise itself up to nearly high water level. It is a mixture of salt and fresh water and its level fluctuates in concord with the rise and fall of the tide on the outside sea. These facts show that it has a connection with the sea water in some way or other.

"*Coffer-dam.*—The geological nature of the ground being as above described, it was thought almost impossible to execute the work in the dry, but as the first step for the work it was decided to enclose the dock site with a coffer-dam, whose total length was 1540 feet, and to adopt a pile foundation, beginning from one end and proceeding little by little and finishing the concrete bottom as the work went on. The puddle on the dam reached −38 feet on the reclaimed part and 24 feet on the land part. On completion of the dam, which took place about a year after its commencement, when the water inside was pumped out to −12 feet, a sinking of one section of the dam occurred, while, at the same time, a part of the bottom of the enclosed site was forced up above water and formed a small island, so to speak.

"*Excavation and Well Sinking.*—Such being the case, it was almost impossible to proceed with the original plan of working even if repairs were made to the dam, and it was decided to execute the excavation, piling, concreting, etc., all under water. As the first step in this task, 15 cylinders, each 12 feet in diameter, were sunk, 7 in front of the entrance, 3 along the north entrance side wall and 5 along the south entrance side wall. Those sunk in front of the entrance, were taken off (two of them partially) to form the entrance after the completion of the dock. Subsequently a row of 6 cylinders, each 10 feet in diameter, were sunk in front of the row of 7 cylinders, and the space between these two rows of cylinders was partly filled with concrete (to −20 feet) and partly with puddle to serve the purpose of a dam when the inside of the dock was pumped out for the facing. Eight cylinders, sunk along the side walls, were embedded in the concrete and formed a part of the wall. The space along the dock head, being very much limited owing to the public road, seven more cylinders were sunk along the head, to serve the purpose of retaining earth. The depths to which these cylinders were sunk were −49 feet to −53 feet for the entrance part and −48 feet to −49 feet for the head part.

"The cylinders were of composite construction of wood and brick,

the lower 24 feet (18 feet in 10-foot cylinders) being made of wood, and the upper 24 to 29 feet of brick. They were all filled with concrete after sinking was completed.

"While the cylinder sinking was executed on one hand, excavation was carried on on the other, which was all done by a Priestman dredger and steam winches. The depth of the excavation was −41 feet along the center line, gradually rising to −38 feet at the sides; where the pumping chamber and rudder well came, they were excavated to −43 feet. The section No. 1, Fig. 436 shows the form, when the excavation was completed; this section also shows the general arrangement of the geological strata.

"*Pile Driving.*—After the excavation was finished, as the necessary preparation for pile-driving, a temporary staging (Fig. 436) was erected all over the site, the posts of which were driven at a distance of 12½ feet both ways. Beams and cross beams being fixed on these posts, rails were laid longitudinally, on top of which frames for supporting pile drivers ran. Rails being laid on the upper face of these frames, pile drivers were able to move transversely; thus pile drivers could be moved both longitudinally and transversely with respect to the dock. Nine drivers were used, and for their working 11 steam winches were set along the north side of the dock site.

"The lengths of the piles ranged from 22 to 25 feet, though at special places piles of over 30 feet were used, and their diameter at the smaller end was 8½ inches. They were driven 2½ feet center to center both ways. In addition, close piles of similar size were driven all round the site to provide against the escape of silt. They were all of pine and the total number of both foundation and close piles was a little over 10,000.

"The rails on top of the staging being laid just at about high water level and the lengths of the piles being such as above sated, it became necessary to use false piles for driving, which were 38 to 41 feet in length. Where timber is abundant such an awkward procedure of using false piles might not have been adopted; but here long piles are comparatively scarce and consequently dear, which condition led to the adoption of the method above mentioned. Of course the use of false piles gave great trouble in driving, both from frequent breakage of the piles themselves and from their connections. The weights of the hammers used were from 1700 to 1900 pounds, and their fall was generally fixed at 10 feet, and the final penetration ranged from ⅛ to ⅜ inch. The total average of number of piles driven per day per driver was 7.9; the minimum being 1, when men were not accustomed at the beginning, and the maximum 15.

"Although extreme care was taken in driving piles, it was rather difficult to judge of their bearing power, especially as they were driven with the use of false piles, and it was thought prudent to appeal to the direct trial. Such trials were made at two random places by loading 100 tons—necessary reduction being made for the buoyancy—of pig iron on top of a wooden frame which stood on four piles. But although each pile had to bear 25 tons net, there was no appreciable settlement in any one of the piles at these places, while the maximum calculated load the piles would have to bear was 11 tons. The section No. 2, Fig. 436 shows the staging, frames, drivers, etc.

"*Rubble Packing.*—After the piles were driven, their heads were cut off and rubble stone was thrown in between them to the average thickness of 3 feet, leaving 1 to $1\frac{1}{2}$ feet of pile heads projecting above the rubble to be subsequently covered with concrete. When the rubble was well rammed in between piles by men on boats and with the aid of divers, it was found that soft silt oozed through the interstices of the rubble and settled on the top of it. This silt was so soft that it could not be removed with any kind of vessel, and it had to be sucked up with the aid of centrifugal pumps. Thus the ground was ready for concreting. The section No. 3, Fig. 436 shows the form at this stage of the work.

"*Concrete Deposition under Water.*—The injurious effect of sea water on concrete seemed to Japanese engineers as quite serious. As concrete was to be deposited in its most unfavorable condition, namely, directly after its preparation, it was decided to change the water inside the dam to fresh water. Although the column of water present in the dock site at that time was calculated at 80,000 tons, as it must be done gradually, over 310,000 tons of water from Kobe waterworks was required for this interchange of sea and fresh water, the latter entering from the surface, and the former being pumped out from the bottom, together with the scum produced by an unavoidable washing of mortar. The proportion of salt to fresh water was ascertained from time to time by analyzing water taken from the surface, middle and bottom at three places, and by finding out the quantities of chlorine and sulphuric acid it contained. The quantities of chlorine and sulphuric acid was 16,869 and 1.924 grams per liter at the commencement, but was reduced to 0.352 and 0.071 grams, respectively, at the end. The charging of fresh water gave also the advantage of maintaining the water level inside the dock site, so as not to injure the concrete in its imperfect state of hardening.

"The deposition of concrete under water was started from the

entrance side in the whole width of the dock bottom and to an average depth of 9 feet, which depth was previously ascertained by experiments. This deposition was accomplished by skips and cranes set on two pontoons, each carrying two hand cranes, men standing on the boats and on banks giving necessary directions as to the proper positions where skips were to be lowered. The skips were made of iron and had a capacity of 32 cubic feet. They were provided with canvas covering to minimize the washing of mortar during the sinking.

"In order to secure the best possible union between the concrete, the work was pushed day and night without interruption. The divers were not allowed to disturb unset concrete and pumps were employed to take off the scum produced by unavoidable washing of mortar. The proportion of concrete for the bottom was 1 part mortar to 1 part gravel, and the mortar consisted of 1 part cement, 1 part puzzuolana, 0.19 part lime and 3 parts sand. The concrete used for the side wall had the proportion of 1 part mortar to 1½ parts gravel, and the mortar consisted of 1½ parts cement, 1 part puzzuolana, 0.25 part of lime and 4 parts sand. The concrete was mixed by three Carey-Latham concrete mixers of 10 cubic yards capacity, and the mortar was prepared by 20 mortar mills of 6 feet 6 inches diameter. The setting time of mortar to be actually used was constantly observed in the cement testing room by immersing mortar in the water taken from the dock site, and its beginning ranged from 8 to 10 hours in water. The utmost care was taken in deposition to join new concrete to old, before the latter began to set. Fig. 436, section No. 4, shows the form of the bottom as actually determined by soundings.

"After the bottom concrete was all finished, the next step was the deposition for side walls, and before it was commenced all dirt was taken off from the bottom concrete surface; minor dirt, scum, etc., were blown off by jets, and necessary frames to confine concrete to the designed form were erected by divers.

"Concrete for side walls was deposited in two layers of 7 feet and 12 feet deep, the same care being taken for deposition as for the bottom. The sections show the form of concrete for side walls, and the necessary frames erected for its deposition. This brought up the top of the concrete to 8 feet.

"Concreting under water was stopped at this level of 8 feet below high water, as by the previous experience it was certain that water inside the dock site, owing to the existence of the outer cofferdam, with the sunken part repaired, could safely be lowered to at least 10 feet below high water level.

"English cement was mostly used, supplied as follows: J. B. White & Bros., 3731 tons; Knight, Bevan & Sturge, 2798 tons; Mikawa Cement Co., 550 tons. The factory of the last-named company is in the Province of Mikawa, Japan.

"Puzzuolana was obtained from one of the Goto Islands (not far from Nagasaki), in the Province of Hizen. Gravel and sand used were mostly obtained from the sea coasts in the vicinity of Kobe. The total quantity of concrete deposited under water was 27,200 cubic yards, and the greatest quantity lowered in a day (24 hours) was about 640 cubic yards. The total number of men, divers, carpenters, engine drivers, coolies, etc., employed for this work only was 149,000 reduced to a day's work of 10 hours.

"*Puddle and Partial Filling.*—The next step taken was to put in clay puddle directly on the back of the side walls all around the dock. Previous to the laying, all the frames hitherto constructed at the back for the deposition of the concrete walls were taken off, and new frames were erected in their places to provide for the bulging out of the puddle. The puddle was prepared by two pugmills driven by steam engines. The prepared puddle from those mills was made to fall into skips (same skips used for concrete) on boats lying alongside the bank, and was lowered into its destined spots, necessary precautions being taken to insure a water-tight joint with natural bed of silt. The actual thickness of the puddle became much greater than the designed thickness of 6 feet, and in some special places it became even about 8 feet, owing to its bulging, in spite of the existence of frames to sustain it and precautions taken to fill in sand as soon as possible to counteract its pressure. The clay used for puddle was obtained from Awaji Island.

"The back filling of sand was carried on together with the puddle laying, care being taken that the surface of the sand should always be below that of the puddle, lest sand should find its way through interstices of planking into the space for the puddle. Puddle and back filling were temporarily stopped when they reached nearly the same level as that of concrete deposited under water. This stage of the work is shown in Fig. 436.

"*Pumping, Concreting and Temporary Loading.*—The interior of the concrete box, so to speak, was thus shut up from the outside except above 8 feet below high water level, and the water inside the coffer-dam was begun to be pumped out. When the water was lowered to −8 feet the top of the side wall appeared above the water, and it was further lowered to −9 feet on the outside of the dock and to −12 feet in the inside.

"As the work was so constructed that there should not be any direct connection between the outside and inside of the dock, it was evident that had there been no fault either in concrete of puddle, there should not be any change in the water level inside this box. Several days' observations showed that the daily increase was only about $\frac{5}{8}$ inch, and this confirmed the belief that there was no appreciable leakage. Consequently had there been no upward pressure to lift the box or had the box been heavy enough to overcome that pressure in case such existed, the box would now have been in a stage to be safely emptied. Careful calculations, however, showed that it was not safe to do so, as the box was not heavy enough to counteract the uwpard pressure of water existing under the bottom. This pressure under the bottom was ascertained by the level of water inside 8-inch iron pipes, which, to provide for the case when the observations of the bottom pressure should become necessary, were previously imbedded in the rubble packing under the bottom concrete. These pipes had open ends and through the bodies of the last ones holes were perforated to make the ingress of water easier. They ran under the side walls and went up through them, and their upper ends reached above high water level. There were six such pipes, one at the head, two on the north side and three on the south side. The observations of water level in those pipes showed that water not only rose quite high up in them to nearly high water level but undulated in concord with the undulations of the external tides, the only differences being in smallness of range and lateness of time.

"The pressure existing under the bottom having thus been ascertained, concrete was further raised on the side walls in the dry (Fig. 436) and, as this alone was not heavy enough, rubble and gravel were thrown inside the dock to serve as a temporary load. The total amount of rubble and gravel was 8000 cubic yards, and thus the excess of the weight of the box above the bottom pressure became more than 12 tons per lineal foot of the dock length.

"All the precautions which were deemed necessary having thus been made, water was pumped out from inside the dock, and it was found that the leakage in the whole dock amounted to only 1 cubic foot per minute, or less than two tons per hour.

"*Masonry Facing.*—Such quantity of water being almost nothing, the preparations for masonry work were at once started, and as the first step for that, a temporary scaffolding, which was to serve two purposes, of lowering concrete, mortar and stone, and of taking out the temporary load, was erected over the center line of the

dock, and after its completion the stone setting was commenced. Four cantilever cranes, which were designed and constructed in the dockyard, were made to run on the side walls; four derricks worked by steam, two 3-ton hand cranes, and two trussed beams, which were spanned between the central scaffolding and the side walls, were the principal machines used for lowering materials, setting stone and taking out the temporary load.

"As the temporary load, above alluded to, was put in to compensate for the insufficiency of the weight of the concrete box, it could not be taken off at once, but had to be removed gradually as the stone facing progressed both on the bottom and the sides. This gave great trouble for working, as the removal of the temporary load, leveling of irregularities of bottom surface with concrete and stone setting must all be done in a very limited space. Stone setting was commenced both from the entrance part and the head, and the greatest number set in a day of twelve hours was 1310 cubic feet.

"The facing stone was all of granite, mostly from Tokuyama quarry, and its thickness was from 1.3 to 2.6 feet along the sides and from 1.5 to 2.5 feet along the bottom. Special care was taken in building the entrance part, large-sized stones being used, and chains and rails being imbedded in suitable positions along the bottom and the sides. Mortar used for stone setting for the dock body had the proportion of 1 part cement, $\frac{1}{4}$ part puzzuolana and $2\frac{1}{2}$ parts sand, and that for the entrance part had the proportion of 1 part cement to 1 part sand, both by volume. The only structures in connection with this dock in which granite was not used were the part of the culvert leading to the pumping house, penstock chamber, and the arch of the lower chamber of the pumping house, all of which were lined with hard burned bricks.

"On each side of the central drain 5-inch iron pipes were laid with branches of $1\frac{1}{2}$-inch iron pipes, which were to collect leaked water, though very slight, and discharge it to the rudder well.

"*Pump House.*—After the bottom concrete was deposited and set hard, a frame corresponding to the inner dimensions (allowance being made for facing) of the pump house was weighted down onto the botton duct, an outer frame was erected outside of it leaving a space between them, which had to be filled with concrete under water and had to form the body of the wall. After the wall reached the proper height, the inside and outside frame were taken off. The inside was then faced with stone and the outside was backed with clay puddle. In that part of the side wall, where the culvert leading to the pump house had to pass, another frame was set down

at the same time with that for the pumping house, and when, after the water was pumped out from the dock, it was removed, a tunnel was formed connecting the inside of the dock with that of the pump house, which tunnel was subsequently lined to form the culvert.

"*Caisson.*—The caisson is of the box-shaped type with four sluices of 20 inches diameter for letting in water, and an electrically driven 5-inch centrifugal pump is set up on it to give greater convenience for the removal of water ballast.

"*Pumps.*—The main pump is an electrically driven 30-inch centrifugal pump, the electricity being supplied from the main electrical station of the company. This is the first instance of the erection of a motor for this purpose in Japan. Besides this main pump, there are an an 8-inch drainage pump and an air-pump for starting the main pump, both of which are driven by electricity. This main pump has the capacity to raise 5000 tons per hour and will lay dry the dock in about four hours. The pumps were those manufactured by the Lawrence Machine Company, and the motors were by the General Electric Company. All these were supplied by the American Trading Company. As there is no need of boilers, the pump house is very simple, its roof standing up only about 2.6 feet above the ground level.

"The dates at which the several works above described were commenced are:

First coffer-dam	November,	1896
Excavation	November,	1896
Cylinder sinking	March,	1898
Piling	December,	1898
Concreting	April,	1900
Stone facing	July,	1901
The dock opened	June,	1902

"The total cost of the work was 1,700,000 yen ($850,000).

"The writer of this paper acted as Chief Engineer for the work, and his chief assistants were Samuru Maruta, M.E., and Jinzo Okamura. The drawing (Fig. 438) shows the construction of the dock and the completed structure."

The construction of the Chanoine dams on the Great Kanawha has involved the building of locks for navigation, for three of which the author constructed the metal work as Chief Engineer for the fabricating company. The foundation for the dams is similar to

the one illustrated in Fig. 439, the coffer-dam for which has been described in Chapter II, and illustrated in Fig. 12. The foundation for the locks is shown in Fig. 440, and the following description has been taken from the *Engineering Record*, the work having been carried out under the Corps of Engineers, U. S. A., by Addison M. Scott, M. Am. Soc. C.E., the resident engineer in charge.

"The Great Kanawha River empties into the Ohio River at a point 262 miles below Pittsburg and 205 miles above Cincinnati,

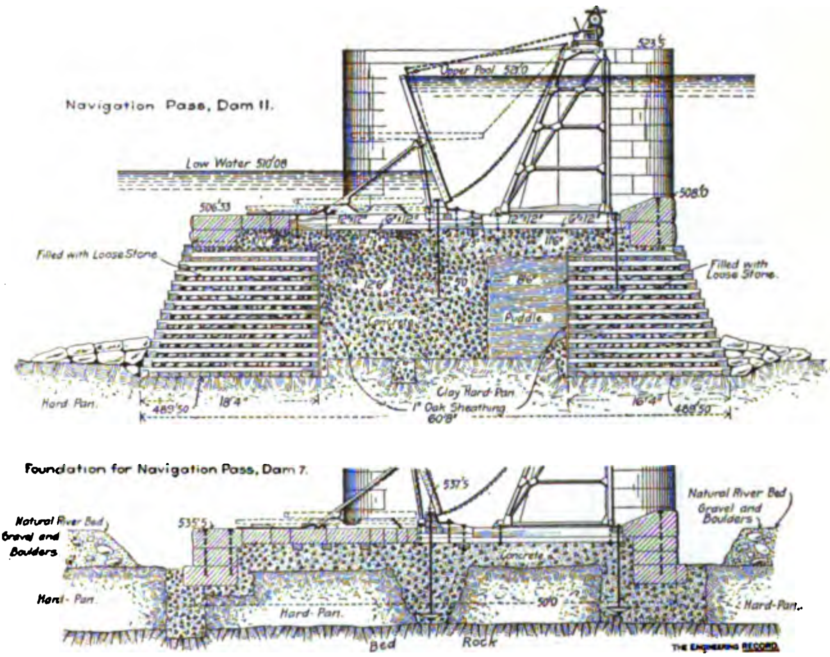

FIG. 439.—KANAWHA RIVER DAM FOUNDATION.

and is a continuation of the New River which rises in North Carolina between the Blue Ridge and Smoky Range.

"The bed of the river is composed of boulders and gravel, with some sand and mud, getting finer toward the mouth. It is underlaid with rock at depths of 7 to 18 feet below low-water mark. The banks are from 35 to 50 feet high, composed mainly of heavy clay, but with frequent mixtures and strata of sand. Ordinary floods rise about 30 feet above low-water mark on the upper portion of the river and about 40 feet near the mouth, while in the lower portion an extreme flood in 1884 raised the level over 60 feet. The

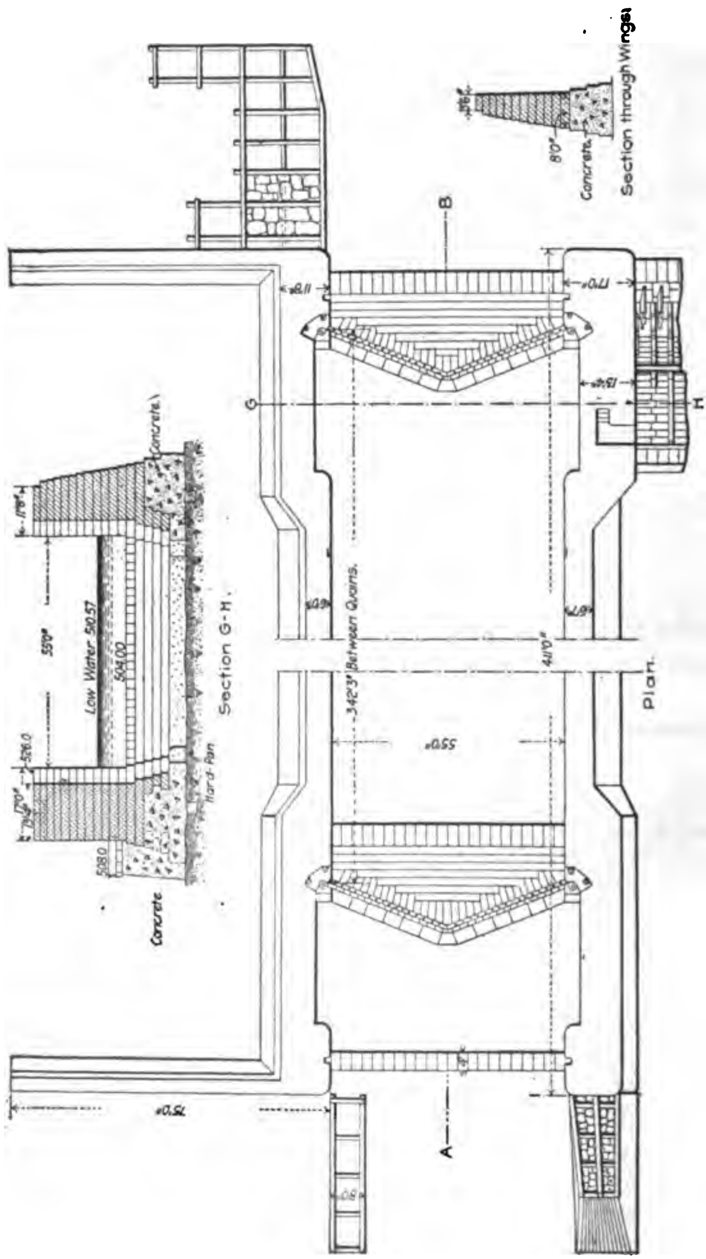

FIG. 440.—KANAWHA RIVER LOCK FOUNDATION.

average width of the river at low water is about 600 feet. The natural low-water depth between shoals is about 3 to 8 feet, but in places it is much deeper. In extreme low water the depth on some of the shoals was originally but a few inches, hardly enough on the shallowest to float a loaded canoe or skiff.

"The movable dams are of the Chanoine wicket type, operated from trestle service bridges. In general features they are all alike. They are easily and rapidly maneuvered, the expense of operation and maintenance is but little if any more than with fixed dams and they prove highly satisfactory to the river interests. They are kept up when there is not sufficient water in the river for coal-boat navigation and down at other times. Their advantages over the ordinary fixed dams for a commerce and river like the Great Kanawha are decided, furnishing the benefits of the usual slack water without its most serious drawbacks. With fixed dams everything must pass through the locks, and navigation is entirely suspended when the river is near or above the lock walls. With movable dams the locks are used only when the discharge of the river is so small as to make them necessary. At other times the dams are down out of the way, affording unobstructed open navigation. Experience with them has naturally suggested improvements, and those last constructed have considerable advantages over those first built in strength and durability of construction, facilities for rapid maneuvering and cost of operation and maintenance.

"Lock and Dam No. 7 are located fourteen miles below Charleston and forty-four and one-fourth miles from the mouth of the river; work here was begun in April, 1889, and was completed in 1893. As this work is typical of the movable dams, a description of this one is given somewhat in detail. (See Report of Chief Engineers, U. S. A., for 1892, page 2059, etc.) The bed-rock at this site was from 11 to $15\frac{3}{4}$ feet below low water and was overlaid with hardpan from $3\frac{1}{2}$ to $8\frac{1}{2}$ feet deep. On this was the river-bed of boulders and coarse gravel mixed with some sand and mud. The foundations of the lock, except of the upper cross sill, all extend to solid rock, while those of the dam rest partly on the rock and partly on the hardpan. The lock is 342 feet long between quoins, with a clear width in the chamber of 55 feet. The total length, not including guard cribs, is 411 feet. The walls are uniformly 20 feet above the top of the miter sills, and, including the concrete foundations, they are from 27 to $34\frac{3}{4}$ feet high. The coffer-dam for the lock and guard cribs measured 536 feet up- and down-stream with shore ends 152 and 134 feet long. It was formed of cribs 15 feet wide, 21 feet long

and about 19 feet high, made of round timbers and sunk to the hardpan, from which the top boulders and gravel had been removed by dredging. They were then filled with heavy dredged material, sheathed on the outside and banked with clay and gravel. For the foundations the hardpan was excavated to the solid rock and replaced with concrete up to within $5\frac{1}{2}$ feet from the top of the miter sills, where the stone masonry begins. Under a portion of the miter sills concrete was not used and the stone masonry was carried to the rock foundations for the purpose of anchorage.

"The stone used on this lock and dam was yellowish and bluish-gray, medium and fine grained sandstone obtained from quarries along the river. It weighs about 150 pounds per cubic foot and the crushing load of 2-inch cubes varied from 25,000 to 46,000 pounds. The chamber faces of the walls are of pointed-face ashlar and the other faces generally of rock-face ashlar except the back of the land wall. The corners, quoins, sills and coping are of bush-hammered dimension stone. The backing of all the walls is of sound, good-size, vertical-sided stones, shaped up and bedded top and bottom and the spaces between the stones filled solid with small stones and spalls. The concrete was mixed in batches made of 33 cubic feet of broken stone, 15 cubic feet of sand and two barrels of Rosendale cement, making about 36 cubic feet of concrete rammed in place. For drainage back of the land wall, loose stone was placed leading to a culvert in the lower wing. To guard against communication between the pools, puddle was placed about the upper wing wall. The miter sills are anchored to the bed-rock with $1\frac{1}{4}$-inch wedge bolts. The lock gates are of white oak built without heel or miter posts, the main beams running through and the ends and center made solid by filling blocks, assembled with horizontal and vertical bolts and keys with the spaces planked. They are suspended at the heel on steel gudgeons and by top fastenings and anchorages, all below the level of the coping. Each leaf complete weighs about $37\frac{1}{2}$ tons. The lock is filled and emptied by valves in the gates, each leaf having five cast-iron valves hung horizontally in a wrought iron frame. The valves are maneuvered by racks and pinions and the gates by spars and capstans. The lock is either filled or emptied at maximum lift in less than four minutes. Steamboats, without tows, are locked either way in from six and one-half to eight minutes.

"Lock and Dam No. 11 are located $1\frac{3}{4}$ miles above the mouth of the river, $56\frac{1}{4}$ miles below Charleston and $88\frac{3}{4}$ miles below the head of slack water. It is at the upper end of a minimum 6-foot channel which connects with the deep water extending to the mouth

of the river and will allow loaded coal barges to be locked down into the natural harbor above the mouth of the river when the Kanawha and Ohio rivers are both low. The importance of this, together with the topography and structures in the vicinity, confined the location to narrow limits and compelled the adoption of one with some unfavorable conditions, of which the depth to the foundations was the most serious. The coffer-dams at this site formed a difficult and expensive part of the work owing to the depth to which it was necessary to excavate to reach a suitable foundation. That for the lock was set upon hardpan between 17 and 18 feet below low water, and extended 12 feet above the water, making a total height of about 30 feet. In order to reach the hardpan it was necessary to dredge from 12 to 18 feet of material, principally fine running sand, necessitating flat slopes.

"This coffer-dam for the lock was built of cribs 19 feet wide filled with dredged material, sheathed on the outside with 1-inch boards and banked with clay and gravel, which were protected from washing by a layer of rough stone. It is 575 feet long up and down the stream, with wings 140 and 125 feet long, making a total length of 840 feet. Some difficulty was experienced in pumping out this coffer on account of part of it not being down to hardpan, but this was remedied by driving heavy sheet-piling outside the sheathing with a pile-driver. The river widens considerably in this neighborhood and consequently the navigation pass and weir are wider than those previously constructed. The lock has the same length and width as all of those below Charleston. With the water below it at extreme low level the lift is 11 feet. The walls are 22 feet above the miter sills and about 35½ feet above the foundations."

The lock on the Lake Washington Canal at Seattle, is next to those on the Panama Canal, the largest in the world. The Lake Washington Canal joins the waters of Puget Sound with the fresh waters of Lake Union and Lake Washington, the distance from Puget Sound to Lake Washington being about 6.5 miles, or from deep water to deep water, about 7.5 miles. Lake Union, in the heart of Seattle has an area of 905 acres, and a depth of 25 to 40 feet, draining an area of 6 square miles. The elevation of this lake was 25.0 feet above extreme low water in Puget Sound, or 7.0 feet above extreme high water. Lake Washington is about twenty miles long, averaging about two miles in width, with an area of approximately 25,000 acres, or about 40 square miles, and portions of this are in the neighborhood of 200 feet deep. This lake receives the drainage of 182 square miles, and receives also the drainage from the 211 square

miles draining into Sammamish Lake, which is about seven miles long, and can be easily connected up with Lake Washington for moderate sized vessels. Lake Washington was 35 feet above extreme low water, or 10 feet above Lake Union, but under the canal the level of both lakes will be about 25 feet above low water in Puget Sound.

The canal was originally planned to have one lock between the Sound and Lake Union, and another between Lakes Union and Washington. The estimate of the United States Engineers for this canal, large enough for the largest vessels, was about $7,000,000. An estimate prepared by the author for a canal with one lock of the lowest possible cost amounted to $3,800,000, and it is presumed that the completed canal will cost in the neighborhood of $4,500,000, including $2,275,000 for the lock.

Borings made a number of years ago by the author, from Puget Sound along the axis of the canal for a distance of nearly two miles, disclosed the fact that very hard blue clay or hardpan could be reached at depths varying up to 25 feet below low water. In the dredging from Puget Sound to the lock site by the author, a distance of about 7000 feet, the fact was disclosed that these borings were fairly reliable, much of the material proving to be so hard that it was just about as cheap to blast and dig it, as to dig it out with a dipper dredge without blasting, not more than 500 yards per day being dug for quite a distance. Some of the material that showed up from the borings to be of the same composition proved to be moderately soft and quite gummy, and very difficult to get out of the dipper. The borings were made with heavy screw bits attached to a square shaft operated in the leads of an ordinary pile-driver. The holes were cased with pipe through the sand and gravel, and by frequently raising the auger good samples were obtained, and the hardness could be closely determined by the speed of drilling.

The excavation on the canal, and the building of the lock is in charge of J. B. Cavanaugh, Major Corps of Engineers, U. S. A., with A. W. Sargent, Assistant Engineer, in charge of the construction of the locks. The contract for the excavation of the lock site and the building of the coffer-dam was let in 1912, at 37.8 cents per cubic yard, round piles 17 cents per lineal foot, sheet-piling and lumber at $27 per M, iron and steel at 5.5 cents per pound, spikes at 4.0 cents per pound and the pumping plant for $8,800.

The coffer-dam was constructed by driving two rows of guide piles 20 feet apart, and 8 feet center to center, having waling of 6×10 timber which was tied together every 4 feet by 1-inch round rods. The walings were spaced 6 feet apart vertically at the top,

$5\frac{1}{2}$ feet apart for the second story, and $4\frac{1}{2}$ feet apart for the lower story. The sheet-piling was of the Wakefield type, made up as described. "Sheet-piles 30 to 38 feet long shall be made of $\frac{2}{3} \times 12$-inch plank with $\frac{1}{4} \times 12$-inch plank in the center, bolted together with $6\frac{1}{2}$-inch bolts; and nailed with $\frac{2}{6}$-inch wire spikes, spaced about every $2\frac{1}{2}$ feet along each side of the pile.

"Sheet-piles 40 to 46 feet long shall be made of $\frac{3}{4} \times 12$-inch plank, bolted together with $6\frac{1}{2}$-inch bolts; and nailed with $\frac{2}{8}$-inch wire spikes, spaced about every $2\frac{1}{2}$ feet along each side of the pile.

FIG. 441.—TEST OF CLAY, LAKE WASHINGTON CANAL.

"Sheet-piles 48 to 52 feet long shall be made of $\frac{2}{5} \times 12$-inch plank with $\frac{1}{4} \times 12$-inch plank in the center, bolted together with $8\frac{1}{2}$-inch bolts and nailed with $\frac{2}{10}$-inch wire spikes spaced about every $2\frac{1}{2}$ feet along each side of the pile.

"Tongues and grooves shall be $3\frac{1}{2}$ inches deep. The center planks shall be dressed to even thickness of about $3\frac{1}{2}$ inches. Wrought iron washers shall be used at the heads and nuts of all bolts." The space between the sheet-piling was filled with the clay from the excavation, which made excellent puddle, the same material being banked up on both sides of the coffer-dam. After the coffer-dam was once pumped out, very little pumping has been done. Eight feet out-

side of the coffer-dam dolphins were driven with intermediate piles, to which boom sticks were attached as a protection for log tows, there being room at the south side of the coffer-dam to maintain a channel 75 feet in width for navigation during construction. The excavation was made within the coffer-dam before it was closed up, to within a few feet of the elevation of the footing courses.

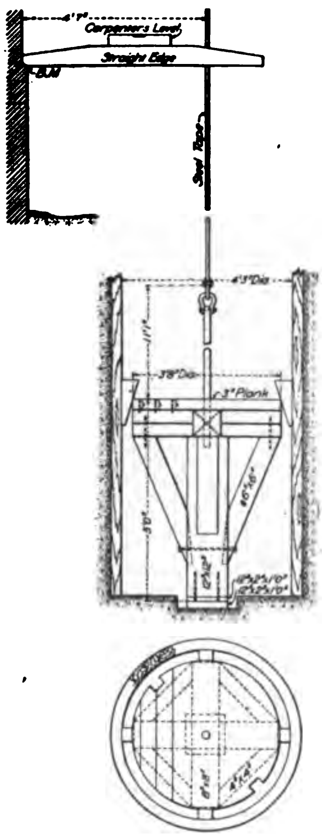

FIG. 442.—APPARATUS FOR TESTING CLAY.

Before construction was commenced on the lock, tests were made (Fig. 441) by loading a platform (Fig. 442) with pig iron attached to a vertical 12×12 timber which was supported by a braced frame-work having a guide around the timber at the top to keep it in line. A load of a few tons was first put on, and levels taken day by day, increasing the load gradually up to 12 tons. A large number of these tests were made and the observed maximum settlement ranged from 0.018 to 0.024 inch and up to a maximum of 0.04 inch. The load to be carried on the clay or hardpan ranged from 3.0 to 4.0 tons per square foot, and from a careful study of the tests it was decided that a factor of safety of from 2½ to 3 would be realized, which was perfectly satisfactory. In making the tests the loose material was removed from the surface and the vertical timber was set on the solid clay. The excavation however was carried in some cases from several feet up to 10 feet below the elevation originally planned, on account of striking beds of sandy clay, it being deemed best in every instance to go to the solid clay or hardpan.

The construction of the side walls of the lock down to the bottom of these footing courses is shown in Figs. 443 and 444, the gantry frames for placing the concrete being also shown in this view.

The walls of the main lock are 80 feet apart in the clear, the length of the lock chamber being 825 feet, divided by an intermediate

Fig. 443.—Lake Washington Canal Lock, General View.

pair of gates into two lock chambers, one 350 feet in length, and one 475 feet in length. The depth of the water over the lower sill at extreme low water is 25 feet, while the regulated depth of the water at the upper sill is 37 feet. A small lock is provided alongside for small vessels, the width being 30 feet, and the length 150 feet. The depth of the water over the lower sill at extreme low water is 12 feet, and the regulated depth of the water at the upper sill is 17 feet.

The records of the construction work are quite remarkable, due both to the very efficient supervision and organization, and to the very complete plant provided. The sand and gravel are delivered at the lock by Government tug and scows from a distance of about forty miles, the cost of the sand and gravel on the scow being 24 cents, and the cost of towing about 14 cents, unloading 7 cents, or a total of about 45 cents. This material is unloaded from the scows

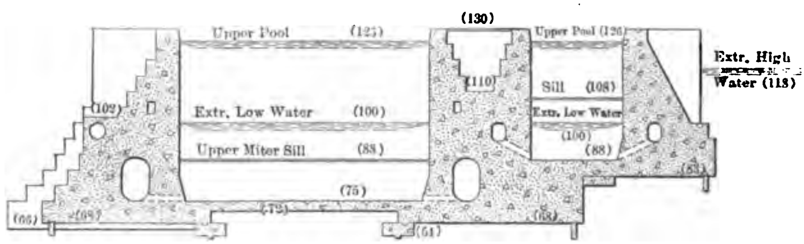

FIG. 444.—LAKE WASHINGTON CANAL LOCK, CROSS-SECTION.

by clam-shell buckets into cars running up an incline to the top of the bunkers, and dumped automatically. The cement of Washington manufacture is delivered on cars at the lock at a cost of $1.50 per barrel. Three 40-cubic feet Smith concrete mixers are employed, with a maximum output of 140 yards per hour, the mixers being operated by electric motors. The greatest output of concrete per day of eight hours has been 1140 cubic yards, and the greatest monthly output has been 23,590 cubic yards, that amount having been reached in September, 1913. With the work about two-thirds completed. the indications are that the average cost of mixing and placing the concrete will be about 38 cents per cubic yard. The forms will range in cost from 40 to 70 cents per cubic yard, with a probable average of 45 cents per cubic yard. The final average cost of the concrete in the lock, including 65 cents for the cost of the plant, will probably not greatly exceed $4 per cubic yard.

The forms for a considerable amount of intricate work, including

the conduits, could not be used over again to any great extent, but the forms for the walls (Figs. 443 and 445) were built in sections as shown, were bolted together at the ends, and supported by bolts set into the concrete. These bolts were located in the forms and extended into the section being poured, by passing through the form plank, and through a board on the outside of the studding in order

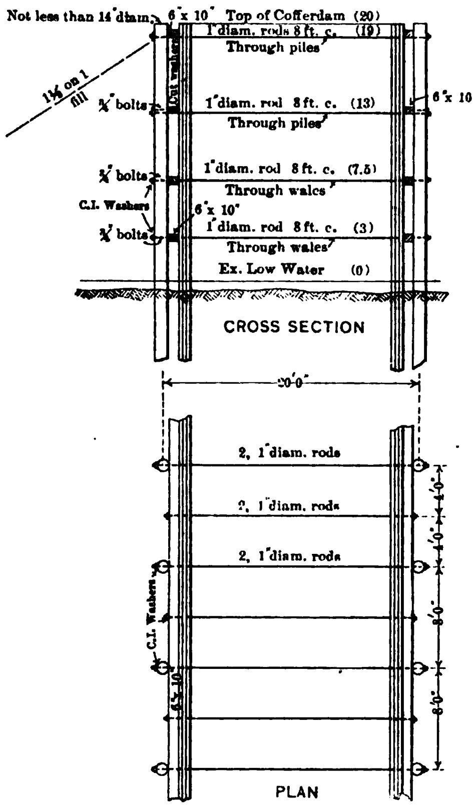

FIG. 444 (*a*).—COFFER-DAM LAKE WASHINGTON CANAL LOCK.

to keep them in proper line. The bolts were screwed into a washer plate at the inner end, and both bolt and washer were coated with tallow so that the cement would not adhere to them. In order to make sure that the bolts could be removed they were given a few turns with a wrench as soon as the concrete had commenced to harden, to make sure that they could be easily turned out later, leaving only the small square plate in the concrete. After one sec-

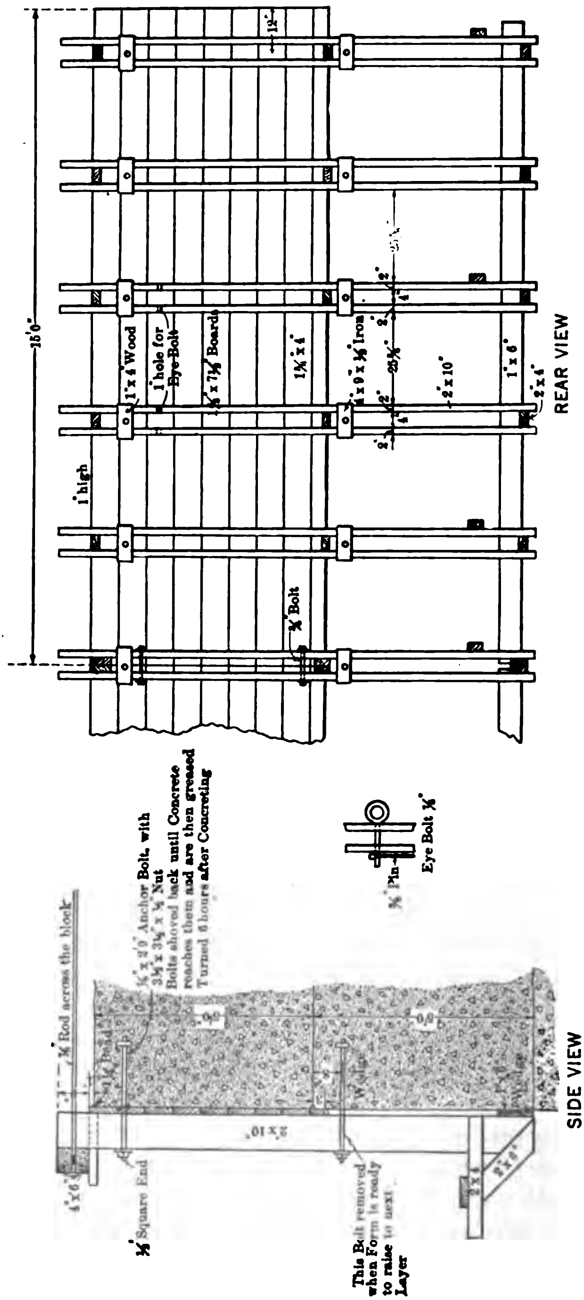

FIG. 445.—FORMS FOR LAKE WASHINGTON CANAL LOCKS.

tion was poured, the sections of the forms were hoisted up by a small sheer leg and bolted in place for the next layer of concrete. The concrete was run from mixers into 1½-yard bottom dumping buckets on flat cars which were switched underneath the gantries by dinkey locomotives. Two buckets were picked up at one time by the gantry, run horizontally over the place of deposit, and lowered down by the operator on the gantry. As the concrete was usually dumped out of sight of this operator, the signal man communicated with him by telephone entirely.

All of the metal work that sets into the concrete is being placed as the work progresses. The bottom or floor concrete will be placed as soon as the wall concrete is completed, and after the lock is entirely finished, a dam will be constructed across the present temporary channel to the south bank of Salmon Bay, which dam will be used as a spillway, and may later be utilized as a portion of a hydro-electric power plant.

CHAPTER XXXI

FORMS FOR CONCRETE

THE construction of forms is very often left to the superintendent on the work and sometimes proves to be the best thing to do, although, usually, if economy is to be fully considered they should be as carefully figured out as other details, and drawings be made in the office from which to build them.

The superintendent and foremen however should be consulted, so that no practical details will be overlooked that will affect the construction of the forms or the carrying on of the concrete work.

The forms for footing courses are the simplest ones to plan, although usually the coffer-dam, crib or caisson acts as the form for the first course, and the upper courses or steps are constructed of 2 or $2\frac{1}{2}$-inch plank set up in panels on the concrete already deposited and braced or shored to the coffer-dam. These forms are nearly always left in, as it would cost more to remove them than the old timber would be worth. Where the coffer-dams and forms are an obstruction in a channel, they must be removed, as was the case with the Salmon Bay piers described in Chapter VI.

The forms for piers are built of timber in the majority of cases, the construction being similar to those illustrated in Figs. 287 and 288 and described in full as they were constructed for the piers of the Red River Bridge. The forms used on the Salmon Bay piers, (Fig. 90) described in Chapter VI were of 2×10-inch surfaced plank, with 4×6-inch studding set 2-feet centers and connected from side to side with twisted wires about every 4 feet, although for a part of the work $\frac{1}{2}$-inch steel rods were used from some stock on hand. Both the wires and the rods were cut off flush with the concrete after the forms were removed.

The forms for the piers of the Vancouver, Wash., bridge where it crossed Hayden Island are shown in Fig. 446, extra heavy studding being used and exterior braces employed to avoid the necessity for tie rods or wires.

FIG. 446.—FORMS ON LAND PIERS, VANCOUVER, WN.

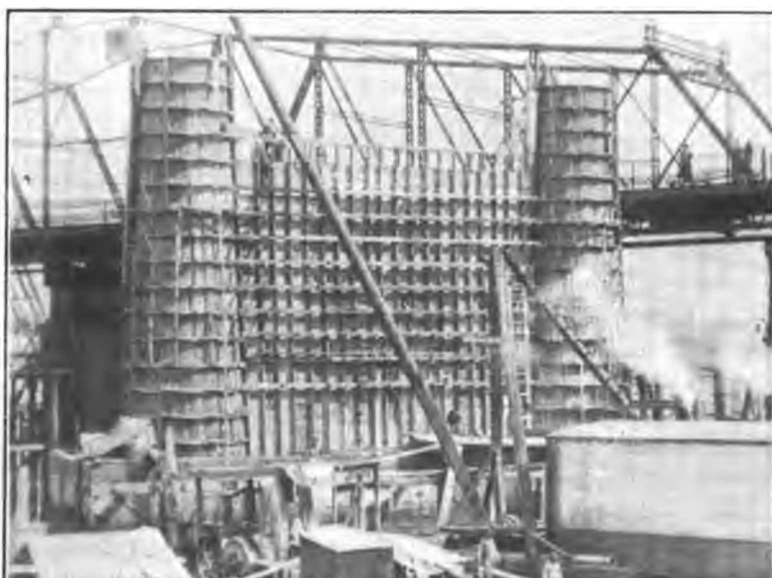

FIG. 447.—FORMS ON ELEVENTH STREET BRIDGE PIERS, TACOMA.

The forms for the author's patent piers on the Tacoma bridge, Fig. 447, are shown in detail in Fig. 448. The rings were made in two layers out of segments of 2-inch plank and braced with No. 9

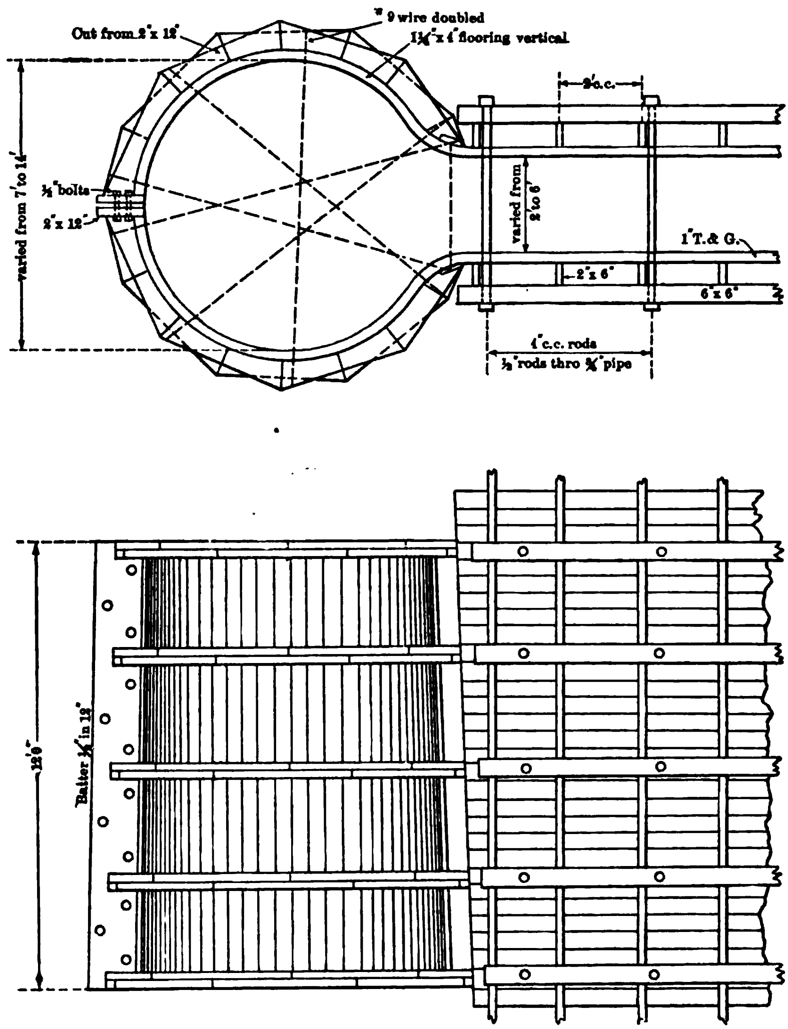

FIG. 448.—PIER SHAFT FORMS, CITY WATERWAY BRIDGE, TACOMA, WN.

twisted wires spaced about 4 feet apart. This enabled the work to be carried on in 5- or 6-foot vertical layers without any bulging of the forms. The sections of the forms for the tapered cylinders were used over again on all four piers, by making extra sections to use

at the top and bottom alternately as the piers were larger or smaller in diameter. Most of the timber from the center portion or web was used over twice and some of it three or four times. The cost of labor on these forms was $1.60 per cubic yard.

The cost of pier forms varies so widely with the form and size of piers, that it is very difficult to arrive at a safe figure except by making a careful estimate of the material and labor.

The cost may be checked by the minimum value of about 80 cents per cubic yard for very heavy pier shafts up to a maximum of about

FIG. 449.—STEEL FORMS, PIQUA BRIDGE PIERS.

$2 per cubic yard for light piers or rather intricate work. The cost of labor alone will usually run from about one-half to three-fourths of the total. The total cost of the forms for the Tacoma piers was $2.10 per cubic yard, which was low for that type of construction on account of the duplication.

Where the forms can be used over for many piers of the same size, metal forms will be found economical and some used on railroad work were described in the *Engineering Record* for Nov. 15th, 1913.

Recent improvements of the Pittsburgh, Cincinnati, Chicago & St. Louis Railway at Piqua, Ohio, include the rectification of about 2500 lineal feet of a single track of the main line by the construc-

tion of a new alignment, about 32 feet on centers from the old alignment, and 18 feet above it, with a provision for second tracking. This section of the road lies along the bank of the Great Miami River, across the stream and through the town of Piqua, where it crosses several streets with overhead plate-girder bridges.

The river crossing is made with five skew spans having concrete abutments and piers on pile foundations, which constitute a heavy

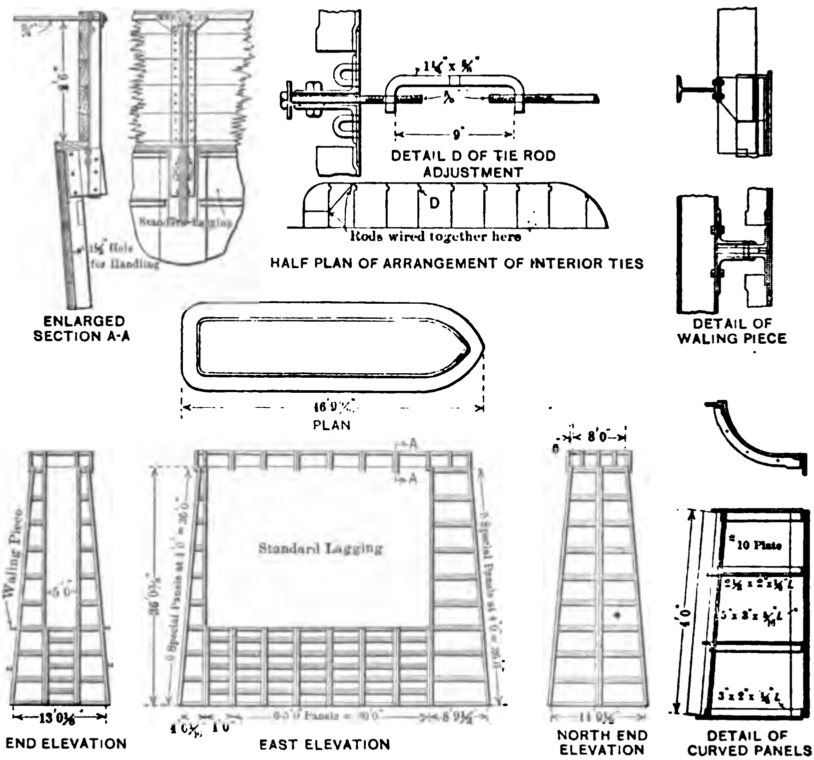

FIG. 450.—STEEL FORM PANELS, PIQUA BRIDGE.

substructure. The east abutment, about 53 feet high over all and 22½ feet wide at the base, has a 78×28-foot footing and makes an angle of about 120 degrees with a wing wall 300 feet long, which follows the river shore close to the water line and serves to retain the heavy fill of the embankment approach to the bridge. The river piers, about 40 feet high and 50 feet long, are 8 feet thick under the coping, and the wing wall has a maximum thickness of 22½ feet above the footing, thus concentrating a considerable mass of concrete at this point.

Excavations for the footings were made below water level by two McMyler 20-ton locomotive cranes, operating a 1-yard orange-peel bucket in the river bottom and a 1-yard clam-shell bucket on land. One of the cranes was equipped with suspended steel leads, 50 feet long, in which a No. 2 Vulcan steam hammer was operated to drive the foundation piles to an average penetration of 18 feet in sand and gravel, about twenty-five piles being usually driven to a refusal of about 1 inch under the last five blows of the hammer in one ten-hour shift by a single machine.

The river piers were constructed in coffer-dams 40 feet wide in the clear, which were built in water about 7 feet deep with a current of one or two miles per hour. Serious floods were not experienced at the time this work was executed, and the coffer-dams were only built to a height of about 9 feet above the bottom of the river. Each coffer-dam consisted of two rows of 2×12-inch vertical sheeting planks spiked to three outside 6×6-inch waling pieces. The rows, 6 feet apart, were connected by ⅝-inch tie rods, 6 feet apart, through each course of waling-pieces, and the space between them was puddled with clay. The bottom of the river inside the coffer-dam was dredged to a depth of about 4 feet and the coffer-dam was unwatered by one 8×6-inch centrifugal and two Emerson steam pumps.

A second coffer-dam was constructed inside the first, concentric with it, and had a clear width of 19 feet. It was made of 2×12-inch wooden sheet-piles, 10 feet long, braced with horizontal struts, engaging two courses of waling pieces. The earth in this coffer-dam was excavated to a depth of 5 feet below the bottom of the dredged pit, and the concrete footings of the piers were laid on the foundation piles, which were driven about 3 feet apart on centers, with their tops extending 18 inches into the concrete. The sheeting served as a form for the concrete. Above the footing course the piers were built in special steel forms. The footings of the retaining walls west of the bridge were built in trenches excavated 8 feet deep by the clam-shell buckets, which generally deposited the sand and gravel excavated from them adjacent to the trench to serve for filling between the walls, thus avoiding hauling the spoil.

The retaining wall was built in wooden forms with sections 19½ feet high and 32 feet long, made of 2-inch square-edge horizontal planks, nailed to 4×6-inch verticals 3 feet apart, with 6×6-inch horizontal waling-pieces 4 feet apart, tied together with ⅝-inch permanent rods having turnbuckle connections to detachable ⅞-inch outside stubs 20 inches long passing through the waling-pieces. Each panel of the forms weighed 15,000 pounds and was handled by

the locomotive crane. Each section of the wall contained about 100 yards of concrete and was made with a vertical groove at one end to provide for a T-joint between successive sections. The coping projected 4 inches beyond the base of the pier shaft, the forms being released from it by slacking off pairs of 4×6-inch wedges on which they were seated.

The piers for the river bridge were built in steel forms made up of panels from about 2 to 10 feet long and wide, bolted together through outside flanges and tied through the concrete with permanent horizontal transverse rods to resist the thrust.

Each panel was made of a No. 10 gage web plate with light vertical and horizontal stiffening and flange angles, riveted to the outer face with $\frac{3}{8}$-inch countersunk rivets. Special curved panels reaching from the top to the bottom of the pier were provided for the rounded up-stream end and for the corners of the down-stream end, and the spaces between on the long side and on the down-stream end of the pier were filled in with 2×5-foot rectangular standard panels, of which enough were provided for two courses 6 feet high, thus permitting the lower course to be removed and reassembled on the upper course, and so on as the work progressed.

Each course was provided with a horizontal 6-inch I-beam waling, bolted to connection angles riveted to the standard uprights of the panels. The uprights were made of pairs of angles with their outstanding flanges separated about $\frac{3}{4}$-inch to receive $\frac{5}{8}$-inch bolts 10 inches long, with the outside nuts bearing on saddle-plates across the flanges of the uprights. The inside ends of the bolts engaged the ends of flat adjustment bars, which were bent 90 degrees at each end and threaded. The opposite ends of the flat bars engaged the left-hand threads of the permanent $\frac{5}{8}$-inch tie rods. Holes punched through the flat bars received the points of capstan bars, by which the adjustments were made, pulling the forms up close against the inside braces, which were removed as the concreting progressed.

After the concreting was finished the short bolts were unscrewed and withdrawn, leaving the connection pieces and the long bolts permanently embedded in the concrete. At the down-stream ends of the piers tie rods parallel to the longitudinal axis were hooked to the transverse tie rods in the second course from the bottom and had diagonal extensions to the forms to resist the longitudinal thrust without the necessity of carrying the rods through the full length of the pier. The forms for one pier were interchangeable with the other piers and were used four times each for the special end sections and sixteen times each for the standard intermediate sections, thus

effecting the entire pier construction with a minimum number of forms, some of which were, after the completion of this job, available for future work.

Connection angles were riveted to the upright flanges of the forms at the top of the pier shaft to receive pairs of vertical angles, bolted to them to take the bearing of the horizontal wooded planks making the forms for the copings. The panels of the forms were assembled and taken apart by hand without the use of derricks.

The work was designed and executed under the direction of the engineering department of the railway, of which Mr. F. H. Watts is division engineer and Mr. E. H. May engineer in charge. The McKelvy-Hine Company, of Pittsburgh, is the general contractor and the steel forms were furnished by the Blaw Steel Centering Company, of Pittsburgh.

The cost of the few planks and timbers needed for the forms for pier footings and the small amount of labor needed for setting the same, will very often amount to a total of as low as 20 cents per cubic yard and should seldom run over 50 cents per cubic yard.

There are many things which enter into the cost of forms and which make it difficult to estimate with any exactness either the cost of labor or material. The concrete skew-backs for the White Pass arch (Fig. 451) constructed by the author in Alaska is a case very much to the point, where labor and material had to be transported a distance of over 1000 miles and inclement weather encountered. The forms for the concrete can be seen to the left and they were not removed until the following spring. Both the concrete work and the steel work were carried out by the author for E. C. Hawkins, Chief Engineer and General Manager of the White Pass and Yukon Railway.

The forms for the East Twenty-First Street reinforced concrete viaduct in Portland, Oregon (Fig. 64), are shown in Figs. 452 and 453. The viaduct was constructed by the author from the designs of H. W. Holmes, M. Am. Soc. C.E.

The abutment forms, Fig. 452, were built of 2-inch lumber surfaced on all four sides, while the studding was of 3×6 scantling spaced 3 feet centers and tied across with double twisted No. 9 wires every 4 feet. The interior of the forms were braced with 4×6-inch struts as shown and the reinforcing was then placed and wired together. The cost for the material was 24 cents per cubic yard, for labor 78 cents per cubic yard or a total of $1.02 per cubic yard. The cost of the lumber was $12 per thousand, labor of carpenters $3.50 per day and laborers $2.50 per day.

The abutments at Knoxville, Tenn. (Fig. 454), are not so heavy

and have long wing walls, but the cost of forms was only 80 cents per cubic yard, the cost of labor and material being practically equal. Wages, however, were much lower than at Portland, so that the higher cost of the two would be a fair guide for preliminary estimates, and for the exact cost in any given case a careful estimate of material and labor should be made. The Knoxville parapets or

FIG. 451.—WHITE PASS AND YUKON RAILWAY ARCH SKEWBACK FORMS.

concrete railing, Fig. 455, are of a very good design for economy, as the forms are very easy to construct, and by making the forms in sections to use over several times, the total cost for both lumber and labor should not exceed $1.50 per lineal foot of railing.

The forms for the floor system, beams and girders of the Portland Viaduct were shored from the ground and from the spread column footings. The forms, Fig. 453, were built from 2-inch lumber surfaced on all four sides, while the posts or shores were rough 8×8-inch

timbers, all thoroughly tied and braced together. The cost of the forms was 60 cents for lumber, $3.85 for labor, or a total of $4.45 per cubic yard. The cost of the shores was 98 cents for lumber, $1.74 cents for labor, or a total of $2.72 per cubic yard. An interesting comparison of similar costs for an arch viaduct may be made by referring to the costs for the Sixteenth Street Viaduct in the same locality, as given in succeeding pages, and which show how large an item of the cost per cubic yard the shoring or centering may become.

FIG. 452.—EAST 21ST STREET ABUTMENT FORMS.

The forms for concrete piles of the molded type are shown in Figs. 60 and 66. They are comparatively simple to construct, but should be figured up for each case, for each shape and size of pile. The forms for the corrugated pile are perhaps the most expensive of any to make, on account of the eight-sided cross-section and the strips for the corrugations. Where the piles are larger, these strips can be of uniform size and cost much less. The form for the jet pipe hole in the center can also be saved by using old pipe of the proper size. In this way the total cost of the forms may be reduced to about 10 cents per lineal foot of pile. The forms for a square pile without corrugations would be very much less. The ordinary cost for

labor will run as low as 5 cents per lineal foot and the outside cost should never exceed 8 cents per lineal foot. The total cost should not exceed 8 or 10 cents per lineal foot where the forms are used over many times.

Where large numbers of piles are to be made, or a pile as intricate as the Socket pile, Fig. 379, is to be molded; metal forms should be employed and removed as soon as safe for the concrete and used over again.

FIG. 453.—EAST 21ST STREET VIADUCT FORMS.

The forms for sewer work are very frequently of sheet steel, properly braced and collapsible so as to be easily removed. The same type is sometimes used for tunnels, but ordinarily it is cheaper to use timber forms in sections, as were those employed by the author in lining some tunnels in the Cascade Mountains, for the Tacoma water system. These forms (Fig 456) for a finished size of 6 feet 6 inches×9 feet, had side sections 12 feet long like the ones shown leaning against the sides of the tunnel. When these sides were properly placed for line and level, four ribs for the roof arch were placed and the filling of the sides begun. When the concrete had reached the top of the side forms surfaced, boards 12 feet long were slid in over the ribs and when the center was nearly reached, boards

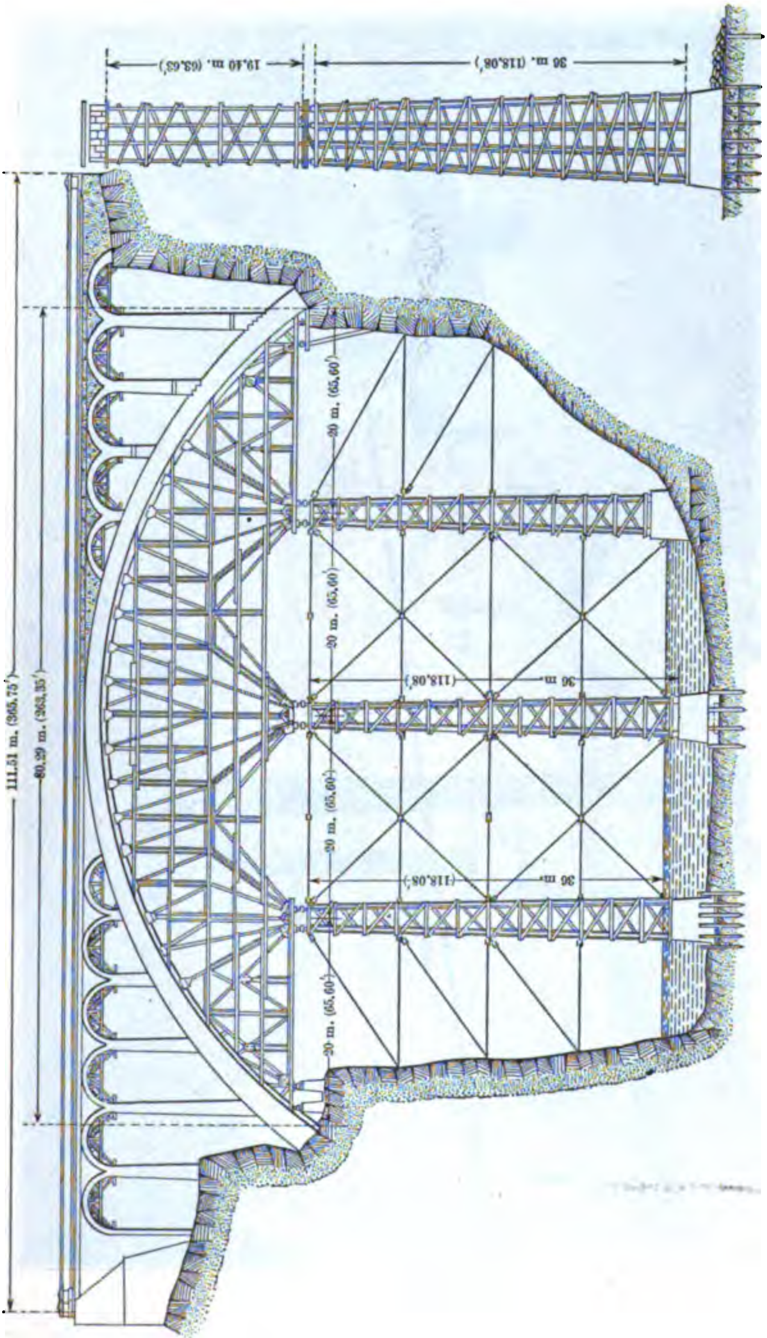

FIG. 453a.—TOWER METHOD OF SUPPORTING FORMS, MONTANGEO, FRANCE.

Fig. 454.—Knoxville North Concrete Abutment.

FIG. 455.—KNOXVILLE CONCRETE PARAPETS.

4 feet long from rib to rib were used, so that the concrete to complete the arch at the key could be more easily stowed away.

The forms for some large pipe Ys and other warped surfaces on the same work, were made up of strips 1 × 1-inch bent around on the ribs in two layers, and such work could not be closely figured up in advance.

Forms, similar to the foregoing types have to be employed for such work as the conduits in the Lake Washington Canal lock walls, the regular forms for which were described in Chapter XXX and shown in Figs. 443 and 445.

The forms for the lock walls for locks in the Illinois and Mississippi canal were described in Chapter XX, and illustrated in Fig. 289.

FIG. 456.—GREEN RIVER TUNNEL FORMS.

These were complete forms for the entire walls, and were braced with 6×6 and 6×8 timbers as shown.

The following account taken from *Engineering & Contracting* gives the construction and cost of forms for concrete lock walls—Lock 21 Cumberland River Improvement, and is condensed from an article by John S. Butler, Junior Engineer, in *Professional Memoirs* for Oct.–Dec. 1911.

The lock walls, guide wall, dam, abutment, and toe walls are of plain concrete, while the curtain wall and footway for the guard wall are of reinforced concrete.

The lock is 386 feet long over all, with 280×52 feet as the effective size of the chamber. The top of the lock wall is 33 feet 8 inches above the concrete floor of the lock chamber. The guard

is 12 feet and the maximum lift, with open river below, is 19.5 feet. The dam which is of solid concrete, 340 feet long has an ogee face on the lower side and a 20-foot concrete apron.

The guard wall, which presents some unusual features, is 160 feet long and is composed of seven concrete piers 10 feet long and 14 feet wide at the base, spaced 12 feet apart in the clear. These piers are joined on their face side with a vertical reinforced concrete curtain wall 3 feet thick and extending from 1 foot below the normal pool level to the top of the guard wall. There is a reinforced concrete walkway 1 foot thick extending over the piers and the curtain

FIG. 457.—GREEN RIVER DAM FORMS.

walls and joining the guard walls to the upper end of the river wall of the lock, an 18-foot drift gap being left between the two structures. It will be seen that the guard wall, although resting on isolated piers, presents, to boats entering the lock, a smooth and continuous surface.

This work was carried on by contract for the first two seasons (1906–1907) during which time, it is said, the contractors lost $100,000. After the annulment of the contract, operations, by hired labor, were started June 1, 1908, under the direction of Maj. Wm. W. Harts, Corps of Engineers, U. S. Army, with the writer in local charge. The cost of the work by hired labor has been close to the contract prices.

While work on the lock was being done by contract, the style of form used for the construction of the concrete lock walls is shown by Fig. 458 (*a*). It will be seen that the trestle which was used for the delivery of the concrete was also used as a skeleton structure upon which the forms were built. The concrete for the lock walls was delivered from a central mixing plant over a 3-foot gage track on the trestle, in steel-bottom dump cars. From the bottom dump cars the concrete was delivered through a hopper car and chute to its place in the forms.

The forms were 6×15-foot panels, built of 1⅝-inch ship-lap lagging nailed to a framework of 3×10-inch pine timbers. The panels were fastened to 10×12 inch posts with ¾-inch lag screws. It was intended that the inside face of the panel be placed flush with the inside face of post, the post thus making part of the face of form; but it was soon found impracticable to keep the posts true to line, and to make a smooth joint between the panels, so the posts were set back 2 inches from the face line of the wall and the spaces between panels built in with a ⅞-inch dressed board. Neither the panels nor the posts were stiff enough to withstand the pressure from the green concrete, so heavy and expensive bracing was required. There was no provision made for moving the heavy panels except by hand, and that proved to be a slow and expensive operation.

After the failure of the contractors and when operations were started at Lock No. 21 by hired labor, the use of the panel style of form was discontinued for the face of the lock walls as having proven unsatisfactory and uneconomical. However, these old forms were used to advantage on the back face of lock and wing walls where extreme care was not required and where derricks were available for moving the panels. When there is sufficient amount of uniform work, the panel style of form, when carefully designed and built, may be used with considerable economy of labor, material, and time, especially when machinery is available for moving the heavy panels.

It was considered best to retain the general arrangement of the concrete mixing and delivering plant for the lock walls, including the delivery track and trestle. The forms were modified with a view of obtaining a smoother and more rigid face, yet retaining the necessary features of the old plan. Fig. 458 (*b*) shows this new arrangement. The sheeting for the face of the wall was built continuously, and for this reason the posts were set back almost 6 inches from the face line of the wall. This sheeting was 2×10-inch pine "S 4 S." The studding was 2×8-inch and 3×8-inch pine spaced from 2 feet 6 inches to 3 feet centers. Old panels were generally

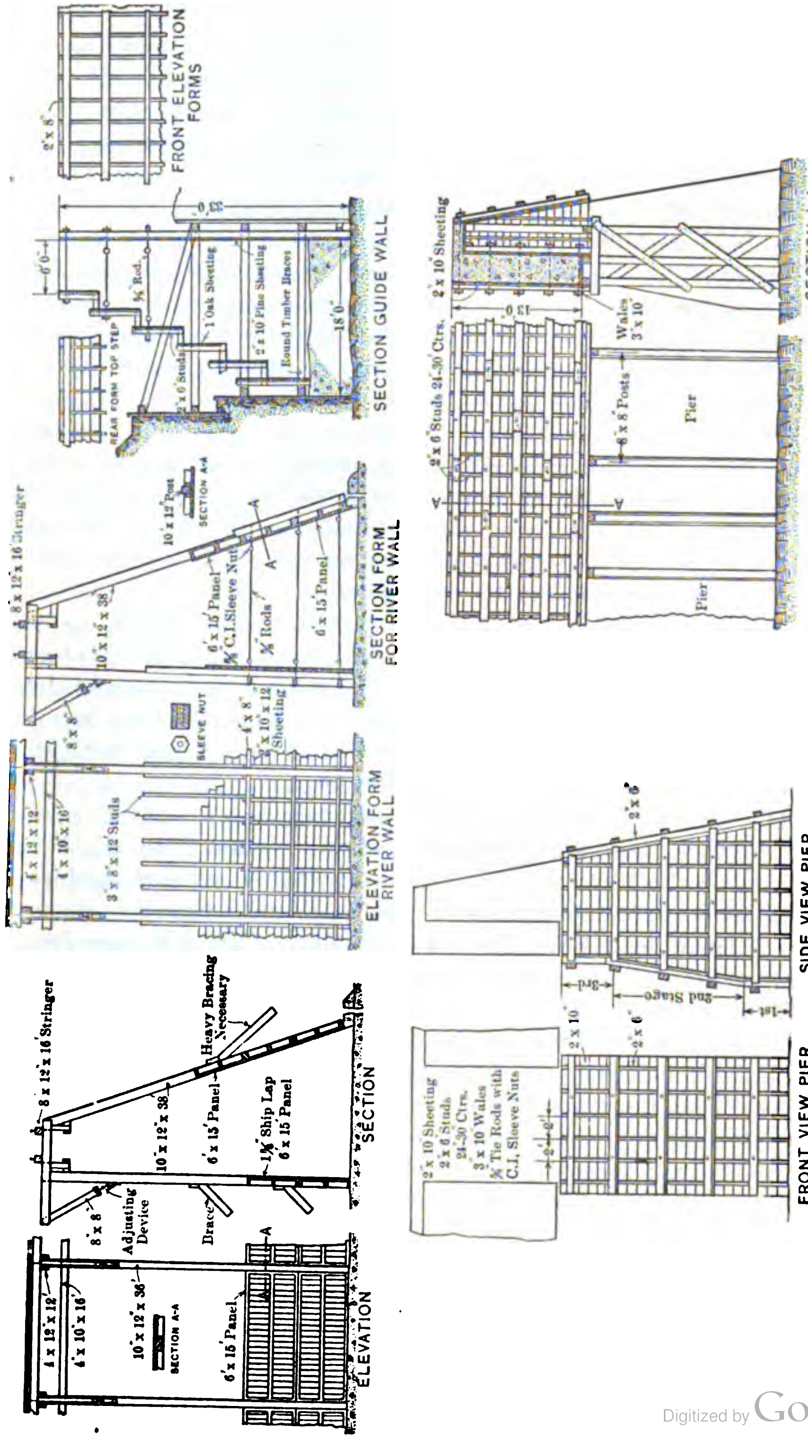

Fig. 458.—Cumberland River Lock Forms.

used for the back of the forms. Tie rods of $\frac{5}{8}$ inch round iron and cast iron sleeve nuts, as shown by Fig. 458 (*b*), were used with very satisfactory results. The rods were placed in the forms only for one run of concrete at a time, so that they would not interfere with the movement of the concrete chute. When lumber became scarce the forms on the lower portion of a section would sometimes be removed before the completion of the top of the section. Generally, the wall was divided into sections 30 feet long. At one time the face form for a completed portion of the wall 100 feet long and 34 feet high was loosened and allowed to fall at one time into the lock chamber, this chamber having previously been allowed to fill with water to break the fall of the forms and prevent injury to the lumber.

On account of the unstable nature of the banks and the large amount of excavation involved, it was decided to use shoring or timber support in making excavation for both the guide wall and the abutment, and on account of similar conditions and plant, the same style of timber support and concrete form was used on both places. This arrangement is shown by Fig. 458 (*c*).

Placing this shoring and bracing was slow and tedious, as was also the removal of the braces while the concrete was being placed. The excavated material was removed by a derrick and placed behind the completed wall for back filling. The concrete was handled from a $\frac{1}{2}$-cubic yard Ransome mixer by a derrick, and with steel dump buckets placed directly into the forms.

The face form was 2×10-inch pine lagging, "S 4 S," carried on 2×8-inch pine studs placed about 30 inches, *c* to *c*. When practicable the form was braced to the timber support, and elsewhere, as near the top of the wall, $\frac{5}{8}$ inch tie rods and sleeve nuts were used. These sleeve nuts, which were about 6 inches from face of wall, permitted the bolt ends to be removed after the concrete had hardened.

These piers were built in the dry by means of a low earth cofferdam. The style of form for the piers is shown by Fig. 458 (*d*). A trestle built independent of the forms was used for delivering the concrete from the central mixing plant in bottom dump cars. Forms were started on the first two piers at the same time, and as soon as the concrete in second stage of pier 1 was completed, the form timbers of the first stage of the first pier were moved to pier 3, and so on from pier to pier.

All piers were completed and forms removed before the forms for the reinforced curtain wall were started. Figure 458 (*e*) shows the forms for the reinforced curtain wall or upper part of the guard wall. The 2×6-inch studding, which was all that was available

for this work, was rather light, requiring a large number of tie rods to keep the forms from springing. Each 22-foot section, 13 feet deep, was completed in one operation, and as this work was done in freezing weather, causing the concrete to set much more slowly than usual, there was an unusual amount of pressure on the forms, and considerable difficulty was experienced in holding the forms true to line. This curtain wall was built when the river was almost to the top of the piers, and as lumber was plentiful and labor scarce, forms were built the entire length of this wall, and the wall completed before any of the forms were removed.

These forms were built of rough oak boxing 1-inch thick and any old material available was used for studs. This oak boxing was seasoned and very uniform in thickness, so a very good face on the wall was obtained. The only trouble encountered was from caving banks and numerous floods. All forms were designed to meet the local conditions as to material on hand, labor, and arrangement of plant for handling concrete.

To my mind the most important feature of economical concrete construction, after considering the arrangement of the plant for handling the concrete material, is the careful study and design of the forms, giving due consideration to the choice of standard lengths and sizes and grades of lumber, with not only a comparison of strengths of the various sizes, but also of the calculated loading to give the allowable maximum deflection. Careful consideration should be given to the minimum amount of lumber required to permit the forms to be kept ahead of the concrete, ample allowance being made for unexpected delay. The lumber should be used over and over as many times as its condition will permit.

To show the relation of the areas, strength, and resistance to deflection of various studs, with the 2×4-inch stud taken as unity in each case, Table I is submitted:

Size of Stud.	Area.	Strength Flexure.	Uniform Load on 60" Fixed Beam to Give ⅛" Deflection.
2 by 4 inches	1.0	1.00	1.00
2 by 6 inches	1.5	2.25	3.37
2 by 8 inches	2.0	4.00	8.00
2 by 10 inches	2.5	5.55	15.60
3 by 8 inches	3.0	6.00	12.00

For the style of forms used and conditions at Lock No. 21, the 2×8-inch stud spaced about 30 inches "*c* to *c*" has been found the most economical.

These styles of forms as submitted herein are not without their imperfections, but they were the best we could devise with the means at hand. They were more economical and gave better results than the forms used by the contractor. The tie rod and sleeve nut were a decided improvement over the timber brace, and the continuous sheeting on the face of the wall gave a smoother and truer surface than was obtained with the panel.

The greatest objection to these forms, as well as to most all forms, is the cost and time required for erection. It will be seen that in some cases a large amount of lumber was tied up, but lumber was at times more plentiful and cheaper than the labor which would have been required to move the lumber from place to place.

The cost of forms for work at Lock No. 21, as given below, seems to be high, but it should be remembered that this includes the cost of all labor and material for erection and removal, including the pro rata of general expense, as well as the additional cost of erection and maintenance of trestle bents and tracks for the delivery of concrete, and also the cost of the shoring and timber supports for the caving banks. The costs were further increased by the extreme care required to obtain and keep a true alignment of the face. Especial care was required in placing the cast iron hollow quoins.

The unit costs of forms for lock walls, including erection and maintenance of delivered trestle and tracks (10,215.7 cubic yards concrete) was as follows:

	Total.	Per Cu. Yd.
Material	$ 2,860.94	$0.28
Labor	5,648.29	.54
Subsistence	961.34	.09
General expense	1,036.35	.11
Total	$10,506.92	$1.02

The unit costs of forms for guide wall, including costs of timber support (2283.9 cubic yards concrete) was as follows:

	Total.	Per Cu. Yd.
Material	$ 765.41	$0.335
Labor	758.49	.332
Subsistence	95.32	.042
General expense	92.33	.040
Total	$1711.55	$0.749

The unit cost of forms for abutment, including cost of timber support (2833.6 cubic yards concrete), was as follows:

	Total.	Per Cu. Yd.
Material	$ 385.85	$0.136
Labor	753.41	.265
Subsistence	184.26	.065
General expense	284.88	.101
Total	$1608.40	$0.567

The unit cost of forms for guard wall, including erecting and maintenance of delivery trestles and tracks (911.2 cubic yards concrete) was as follows:

	Total.	Per Cu. Yd.
Material	$ 454.70	$0.499
Labor	1108.47	1.216
Total	$1563.17	$1.715

The excessive cost of the forms for the guard wall was due to frequent floods delaying and damaging the work, and to increased difficulties of the forms for the reinforced curtain walls.

The cost of forms for the top wall, having a volume of 625.3 cubic yards of concrete, was as follows:

Item.	Total.	Per Cu. Yd.
Labor	$341.55	$0.548

Material, old lumber, not charged.

The methods and cost of constructing East Park Dam, California, Orland Project, U. S. Reclamation Service are given in an article in *Engineering & Contracting* by E. G. Hopson, Supervising Engineer, U. S. Reclamation Service, Portland.

The East Park Dam of the Orland Project of the U. S. Reclamation Service lies on one of the upper feeders of Stony Creek, the principal tributary of the Sacramento River from the Coast Range portion of its drainage area. Like all of the Coast Range tributaries the summer flow of Stony Creek is insignificant, although the winter and spring runoff is frequently excessive.

The dam site is a notch in a great conglomerate dike or ridge extending in a north and south direction, characteristic of the folds of the Coast Range area. The river has cut through this ridge a deeply eroded channel, and during ages has built up its bed with sand and gravel deposits, so that the level of the stream to-day lies some 40 or 50 feet above the originally eroded rock bottom. It was decided by the Reclamation Service to adopt a solid concrete dam of gravity section built on a plan arching up-stream to a horizontal radius of 275 feet. The spillway was located about one-half of a mile from the dam at a depression in the same ridge on which the dam was located. At three other points on the reservoir margin where low places occurred small earth dikes were built. The total cost of all construction work for dam, spillway and dikes was estimated at $198,000, including an allowance of 25 per cent for engineering administration and contingencies.

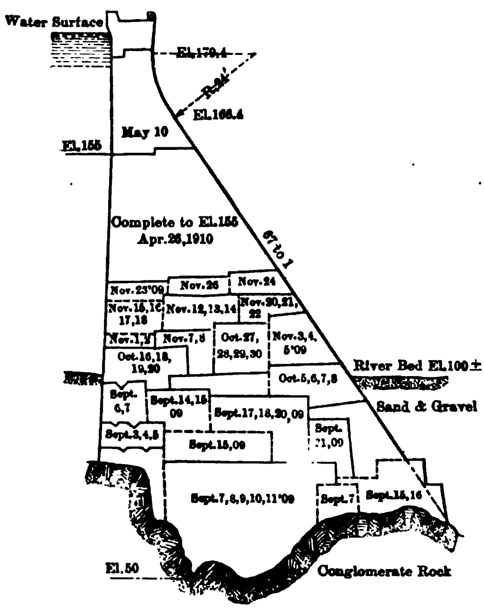

FIG. 459.—ORLAND CALIFORNIA DAM FORMS.

The maximum height of the dam is 140 feet above the foundation rock; the length along the top is 249 feet. Fig. 459 shows the maximum cross-section of the structure and the methods used in building up and bonding the mass of the concrete. Expansion joints running entirely through the structure were built at intervals of 20 feet. Two outlets were provided: One 24 inches in diameter at a depth of 85 feet, and the main outlet, a 4×5-foot gate at a depth of 65 feet below the water surface of the reservoir. No notable difficulties occurred during construction. The principal material excavated for the foundation was sand and gravel. Two pumps—one 5 inches and one 6 inches—easily handled all of the water in the foundation pit. The excavation did not, as was anticipated, disclose a fault or plane of fracture or movement in the conglomerate dike, it being evident that the gorge originated only through water erosion.

The spillway consists of nine semicircular reinforced concrete arches supported by massive abutments. This design was adopted with a view to securing the greatest possible length of spillway with a minimum of construction cost. The spillway length is 430 feet. Its estimated discharge capacity with a depth of 3.8 feet on its crest is 10,000 second-feet (about twice the maximum flood peak on record). The spillway was founded on shale rock and below the structure no attempt was made to protect this rock from erosion, it being considered that although some weathering and erosion might occur and ultimately a masonry lining be necessary, the shale was sufficiently resistant in itself to warrant deferring such protection for some time. The event, however, has disproved this forecast, as on March 7, 1911, the reservoir was entirely filled and about 2000 second-feet of water passed over the spillway. Experience showed that the shale below the spillway was inadequate to resist the erosive action of a large flow, deep channels being scored in its surface up to a point about 50 feet below the spillway structure itself. The United States is now extending the masonry of the spillway for a distance of about 200 feet, terminating the structure on the solid conglomerate rock.

Some facts and cost figures about the dam and spillway construction will probably be of interest, as the job is a compact one free from complications.

All cement was manufactured at Tolenas, California, cost price f.o.b. cars being $1.55 per barrel. The cost delivered at the nearest railroad station to the work was $2.05 per barrel. Cement and all material brought by rail required hauling over 18 miles of mountain road. The average price of hauling cement, iron work and other materials was 32 cents per ton mile. The cost of road haul and storage for cement was $1.08 per barrel, so that the net cost delivered at the work was $3.13 per barrel.

In the main dam the total concrete built was 12,202 cubic yards, in which 12,382 barrels of cement were used, or 1.01 barrels per cubic yard of concrete. The mixture was generally proportioned at 1 volume of cement to 10 of the unmixed aggregates.

In the spillway a richer grade of concrete was used the total yardage being 1456, in which were placed 1758 barrels of cement, or 1.21 barrels per cubic yard. The mixture was generally proportioned at one of cement to eight of the unmixed aggregates.

The concrete was mixed in standard revolving mixers and handled by cars and track.

The principal item of construction was placing concrete in the dam and spillway as given in Tables A and B.

TABLE A—COST OF CONCRETE IN SPILLWAY, 1456 CUBIC YARDS

Items.	Total Cost.	Cost per Cu. Yd.
Cement delivered at R. R. station (1758 bbls.)	$3620.99	$2.487
Cement—hauling and storing	1961.25	1.340
Form—material	373.15	0.260
Form—labor	1418.50	0.980
Sand and gravel—labor and furnishing	2438.80	1.670
Mixing and placing	1388.70	0.960
Finishing	414.40	0.280
Total		7.977
Preparatory expense	$161.35	0.111
Interest on investment	1259.00	0.869
Plant depreciation	318.95	0.218
Miscellaneous and supplies	654.71	0.447
Total		1.635
Superintendence	$1233.41	0.846
Engineering	913.91	0.628
General administration	1503.27	1.032
Grand total	....	$12.118

TABLE B—COST OF CONCRETE IN MAIN DAM, 12,202 CUBIC YARDS

Items.	Total Cost.	Cost per Cu Yd.
Cement delivered at R. R. station (12,382 bbls.)	$25,333.86	$2.076
Cement—hauling and storing	13,394.98	1.097
Forms—material	2,054.39	0.168
Forms—labor	5,143.45	0.424
Sand and gravel—labor and furnishing	7,074.20	0.580
Mixing and placing concrete	5,304.29	0.434
Finishing	429.60	0.035
Total	....	$4.814
Preparatory expense	$1,817.57	0.149
Interest on investment	4,326.00	0.354
Plant depreciation	3,201.31	0.262
Miscellaneous and supplies	6,700.08	0.550
Total	....	$1.315
Stream control and unproductive work at quarry	$2,611.34	0.213
Superintendence	7,530.02	0.617
Engineering	5,800.14	0.475
General administration	9,540.59	0.782
Grand total	....	$8.216

The dam has successfully withstood one season's test of full reservoir. Before water was backed up against the dam, sighting marks were placed on the rock abutments at either end of the structure and on the structure itself for use in determining whether any deflection took place under full reservoir pressure. Careful observations, however, failed to detect any movement.

There was a somewhat elaborate system of interior drainage contrived in the dam structure to carry off any water passing through the expansion joints, and deliver it into the outlet conduit. These drains are to a considerable extent successful, most of the percolating water being intercepted. In spite, however, of the drains and of the care taken in construction to avoid any continuous joints through the dam, seepage water forces its way through the masonry following mostly horizontal plates between the several blocks of concrete. Fig. 459 shows the points where seepage water appears on the downstream face. The amount of this seepage is inconsiderable, usually not being sufficient to maintain a continuous flow to the bottom of the dam. Its most serious defect is discoloration and unsightliness.

The forms for the Green River Dam, Fig. 457, were of 2-inch surfaced plank, spiked inside of the curved ribs as shown, which were made up of two layers of plank cut to form and spiked together break joints. For large dams with curved and warped surfaces, the same design may be employed, but for uniformly curving surfaces or regular battered faces of dams, the type used for the Lake Washington Canal locks would be best.

The time for the removal of forms is a very important matter and the following is taken from an article in *Engineering & Contracting* on concrete forms by Jerome Cochran, civil engineer.

REMOVAL OF FORMS AND CENTERS

GENERAL REQUIREMENTS

Forms shall not be removed until the concrete shall have become hard enough to be unquestionably self-supporting. No forms should be allowed to be removed except in the presence of the inspector. The most important precaution in reinforced concrete construction, and whose importance cannot be overestimated, is *caution in the removal of the form work.*

Notification of Form Removal.—No forms whatever should be removed at any time without first notifying the engineer in charge of the work. But such notification should not be considered to relieve

the contractor of responsibility for the construction and removal of such forms.

Contractor's Risk.—Forms are removed from the concrete at the contractor's risk at any time, and should any of the concrete give way by such removal or become permanently injured, the contractor should be required to remedy same at his own expense. The contractor should be expected by suitable observations to know when the concrete in any section of the work is sufficiently hardened to bear its own dead load plus additional load as may be imposed by the work of installation and should not be relieved of responsibility for premature removal of centers. The engineer, however, may when he deems advisable, order the centering to remain for a longer time. The engineer's acquiescence in permitting removal of forms should not by any means relieve the contractor of responsibility for same.

Time of Removing Forms.—Under no circumstances should forms be removed until the concrete has attained sufficient strength to resist accidental thrusts and permanent strains which may come upon it. Forms supporting reinforced members should be left in place until the concrete rings sound and is readily chipped by a blow from a pick. Much attention must be given to this portion of the work, which is fraught with danger under incompetent direction. No exact time for the removal of form can be safely prescribed because of the varying character of the work, the variations in the setting of different cements and the influence of atmospheric conditions. Forms should, however, remain longer under beams and arches than around columns or walls, and longer under beams and arches of long spans than of short spans.

City Ordinances.—Removal of forms should comply with the city ordinances or regulations governing reinforced concrete construction and forms should not be removed until the approval of the engineer is obtained.

Foreman to be in Charge.—A competent and experienced foreman should be in charge of the removal of forms at all times. At no time are more men to be engaged in the striking of forms than the foreman can fully direct and supervise. No foreman lacking the required experience in this line on high grade work, should be allowed upon the work.

Test Concrete Beams.—As an aid to judging when forms and supports may be removed safely, and when the concrete work may be used safely to carry extraneous weights or loads, test concrete beams about 4×6 inches by 3 feet in length, can be made of concrete taken

from batches which are being placed in the portions of the structure under consideration.

Forms not Supporting Loads.—Forms which do not support loads may be removed as soon as the concrete has taken its final set.

Minimum Time Limits for Form Removal.—(*a*) The minimum time for the removal of forms (not the supporting shores) should be as follows: For walls in mass work, 2 days; for thin walls, 3 days; for sides of lintels, girders and beams, 3 days; for bottom of slabs, 4 days; for spans of 6 feet or less plus 1 day extra for each additional foot of span; for columns, 2 days.

(*b*) The minimum time for the removal of shores should be as follows: For bottoms of beams and girders, 14 days, for spans of ordinary length.

(*c*) Should frost occur before the concrete has attained sufficient strength to enable the removal of the centering, the counting of time for this removal should start after the influence of frost has been entirely eliminated.

Freezing Weather.—When the necessary precautions are taken during freezing weather, the forms may be taken down at the usual time, if, in the judgment of the engineer, the test prisms made under the same conditions as the concrete is made, has attained sufficient strength. As a general thing, however, the forms must remain in place much longer than in warm weather. Special care must be used in removing centering when the concreting has been done in cold weather. Falsework must never be removed while the concrete is frozen. If necessary, artificial heat must be employed to thaw the whole mass and the set and hardness determined before removing false work. In other words, do not remove any forms until absolutely certain that the concrete is thoroughly hardened and that no portion is either soft or frozen. The only sure way of knowing when the cement is fully set and the concrete properly hardened, is to actually test it with a hammer for hardness. To do this, it is necessary to remove small portions of the formwork in each section of the structure to be certain that there are no soft spots.

Forms to be Easily Removed.—Forms should be removed gently without chipping or jarring the concrete. Prying with bars or striking with a sledge must not be allowed as all jar and vibration must be avoided.

Manner of Removing Forms.—Forms when being taken down must not be dropped onto floors and knocked against columns and walls. The forms should always be eased down by a cable or other similar method. Always leave in place a few intermediate posts

so as not to place the entire dead weight too suddenly on the beams and columns. A regular procedure should be followed in removing forms, and the work should be done by regular gangs so that the men become trained in the requirements and methods of the work. Care must be exercised and precautions taken to prevent large masses of forms from falling on floors. In other words, the work of removing the forms, molds, and centering should be done with great care so as to avoid injury to the concrete.

Cleaning and Piling Forms.—The forms upon removal should be thoroughly cleaned of all cement and any necessary repairs should be made and the forms piled in some convenient place.

Superimposed Loads on Forms.—When forms are being removed, there must be no load upon the portion of the concrete affected in excess of one-sixth of the live load for which the portion affected was designed, unless temporary shores are left in to take care of such load.

COLUMN FORMS

Column forms, if removed first, should be so removed as not to disturb the beam and girder forms.

Time of Removal.—Column forms should not be removed in less than two days, in summer; in cold weather, four days; provided girders are shored to prevent appreciable weight reaching columns.

SLAB, BEAM AND GIRDER FORMS

Floor Slabs and Sides of Beams.—Forms should not be removed from floor slabs of ordinary spans in less than seven days. Sides of beams and girders should not be removed in less than three days. The bottom of beam and girder forms should remain in place until after the side forms have been removed so as to inspect the sides of the beam without lessening the support of the beam against collapse. In all cases leave, at least, one line of shores in the center of the floor slabs when removing floor forms.

Beam and Girder Supports.—The original supports for all beams and girders should remain in place at least fourteen days, but all beams and girders having more than 30-foot span from center to center of support should be considered as special cases and should be subject to inspection of the Building Department before removal of supports.

Freezing Weather.—The time at which props or shores may safely be removed from under beams and girders will vary with the condi-

tion of the weather, additional time, however, should be allowed for each and every day that the thermometer registers any time during the day or night below 35° F.

Removal of Shores.—Before removing the shores under any beam or girder, the column supporting it should be stripped, so that columns may be examined on all sides, and at least one side of each beam and girder form should be removed in order to expose the concrete to view, so as to give evidence of the soundness and hardness of the concrete. The shoring underneath principal girders and beams should be the last to be removed.

Method of Removing Shores.—Shores should be removed without jarring the structure by properly pulling the double wedges at the bottom. Shores should be lowered gently and not allowed to drop heavily onto floors and thrown against columns. When shores are finally removed, they should be taken out for a beam or a panel at a time and under no circumstances must all shores under a floor be knocked down at haphazard or in rapid succession.

Removing Shores before Forms.—While it is customary practice to some extent to remove shores one at a time and then put them back again in order to permit the removal of bottom boards of beam forms, etc., this practice should not be permitted. The reasons are obvious.

Precautions to be Observed.—In removing forms the falsework should be lowered to such extent as to permit the form to drop away an inch or two from the slab, in which position it should remain for twenty-four hours. While the wedges are being loosened the concrete must be carefully inspected.

WALL FORMS

Massive Wall Forms.—Forms for massive concrete walls should not be removed in less than one day, or when the concrete will bear pressure of the thumb without indentation. If indented, the concrete is too soft to permit of removing the forms.

Thin Walls.—Forms for thin concrete walls should not be removed in less than two days for ordinary conditions; in cold weather, five days.

ARCH CENTERS

Striking Centers.—The centers should be struck when directed by the engineer, which direction should not be given until the masonry

above them has been completed up to the level of the bottom of the coping.

Time of Removing Centers.—Centers for arches of small size should not be removed in less than one week and for large arches with heavy dead load not less than one month.

Method of Removing Centers.—Arch centers should be removed without shock or jar to the arch ring. Centers should be lowered evenly and gradually, so that the ring can settle uniformly. For very long spans the engineer will provide special instructions for striking centers.

MISCELLANEOUS FORMS

Conduits.—Forms for conduits may be removed within two or three days, provided there is not a heavy fill upon the conduits.

Sidewalk Forms.—Forms for cement sidewalks should be left in place until the concrete or mortar has set.

The costs given for forms, as well as on other items of engineering construction, very rarely give the cost of labor per day for the various grades of workmen, the unit price of materials, or the conditions in detail that existed.

Each engineer therefore should carefully keep such data from his own practice and preface each item in the record book with all the detail that would be necessary for any one to get a full grasp of that particular cost; especially in view of the fact that the conditions may become hazy in the mind of the person making such record, and that wages, cost of material and conditions may change completely within a comparatively short period of time.

CHAPTER XXXII

ESTIMATING THE COST

THE three factors to be utilized in making any estimate are the cost of labor, cost of material and judgment.

The cost of labor is very difficult to arrive at, due to the variation in the rate of wages in different parts of the country or the world, the variation from year to year, the fluctuations from season to season and that due to the amount of work in progress in any particular locality.

The method of arriving at the cost of labor, from unit costs derived from work already executed, is very dangerous, as conditions are seldom the same, and more money has been lost from this practice than in almost any other way.

If unit costs of labor are used from old data, they must be employed with fear and trembling, and only by bringing to bear to the fullest extent the estimator's judgment. Where the unit costs are based on work executed by the person using them, and used on a new piece of work, there is a chance for the judgment to be good, but if such is not the case, some data may have been omitted from another's record, which will make it impossible to form a correct judgment.

Should any uncertainty exist regarding the data of the work or the data of the unit costs to be employed, then the work must be very carefully examined and fuller data gathered on which to base a judgment.

The cost and efficiency of labor have changed completely since 1900 or within the past ten or twelve years. First, the hours per day have been almost universally reduced to the "eight-hour" basis, without a correspondng reduction in the rate, so that if men are employed nine or ten hours per day, the cost of labor is increased from 10 to 20 per cent.

Second, the lesser hours or the higher rate per day has had a tendency to reduce the efficiency of the workmen, and in many classes of work only 50 or 75 per cent of the amount of work is done

per dollar expended as formerly Third, the shortage of labor in certain parts of the world for long periods and the shortage in other parts during the summer or busy season, has a tendency towards inefficiency and an unstable rate of wages.

These things have not always been carefully considered by contractors in making proposals, or at least they are not reflected in the proposals themselves, and contract prices have not kept pace with the cost of labor and material.

The cost of material should be obtained for each particular piece of work, with an option or cover for the time necessary for the proposals to be considered and the award made. Where this is not possible, due allowance must be made for any prospective rise in prices.

The judgment of the estimator must be brought to bear on all these facts, and on everything that has a bearing on the cost, such as the availability and efficiency of superintendents and foremen. the general items to add to the cost, such as plant and general expense.

The entire cost of plant and plant installation and removal must often be included, and if the entire cost of plant is not included, then a very large item for depreciation of plant must be incorporated in the estimate. Second-hand plant can often be purchased, but it must be inspected by a thoroughly competent person.

Plant repairs is a very important item in a cost estimate, but this is usually charged to "Repairs and Renewals," which is included in the "General Expense" item which must be included in every cost.

"General Expense" or "Overhead Charges" is a part of the cost of a piece of work, as much as is the labor and material, and wherever the word cost occurs in a specification or contract it must be interpreted to include the "General Expense," unless cost is specifically defined otherwise. When such is the case the percentage allowed on force account work is stated at a figure that will cover "General Expense" and a profit, usually from 15 to 20 per cent on the cost of labor and materials alone.

General Expense in connection with a contract business will run from 5 to 10 per cent per annum on the total amount of contracts completed during the year. It will include the ledger accounts of general expense proper, contracting expense, office expense, office salaries, insurance, taxes, interest, repairs and renewals, miscellaneous earnings, and any other accounts not chargeable directly to particular contracts.

FIG. 460.—FALSEWORK AROUND TACOMA PIER.

When an estimate has been completed according to the best judgment of the estimator, then it must be carefully checked by someone else to be sure that all computations are correct and that all items have been included.

It has been the author's practice to have a synopsis made of the specifications for a piece of work, as soon as they are received, lists made of all material on which prices must be obtained and a preliminary estimate made as a guide to the exact or final one.

Many tables may be prepared for such purposes, similar to Table LXVII, giving approximate costs of various types of foundations in different kinds of material and to different depths.

TABLE LXVII.—APPROXIMATE COST OF FOUNDATIONS

FOR PRELIMINARY ESTIMATES ONLY

Cost per Cubic Yard of Area of Base×Distance from Top of Base to Bottom of Piles or Crib

Type.	Depth.	Soft.	Medium.	Hard.
Concrete base 6 ft. thick on piles	18	3.50	4.50	5.50
	28	3.00	4.00	5.00
	38	2.50	3.50	4.00
Caissons or wells sunk by dredging (no piles)	30	9.00	11.50	15.00
	40	12.00	15.50	22.50
	50	15.00	18.50	
Coffer-dam pumped to foundation bed (no piles)	20	9.50	10.50	12.50
	25	11.50	14.00	17.50
	35	13.50	18.00	
Concrete under water, plank driven for forms and dredged out (no piles).	20	7.50	9.50	
	25	9.50	13.50	
	35	12.00	18.00	
Pneumatic caissons with concrete filling	40	12.00	18.00	25.00
	60	18.00	24.00	30.00
	80	24.00	29.00	35.00
	100	30.00	35.00	40.00

The above approximate costs are based on the usual or medium-sized piers. Very large ones will cost somewhat less, while small ones will cost considerably more.

The table of preliminary cost of concrete forms, Table LXVIII, is one in which the values will vary greatly for the various parts of the world, and the engineer should prepare a similar revised table for his own particular locality.

TABLE LXVIII.—COST OF FORMS FOR PRELIMINARY ESTIMATES

Cost per Cubic Yard, without any Shoring

Class.	Solid Thin.		Solid Thick.		Reinforced.	
	Labor.	Total.	Labor.	Total.	Labor.	Total.
Pier footings	0.25	0.50	0.15	0.25		
Pier shafts	0.90	2.00	0.35	0.80	1.60	2.40
Abutments	0.90	1.50	0.80	1.10	1.50	2.50
Pedestals	0.20	0.40	0.25	0.50	1.00	1.50
Retaining walls	0.80	1.10	0.50	0.90	1.75	2.50
Seawalls	1.00	1.40	0.70	1.10	2.00	3.00
Wharves	......	......	......	......	4.00	6.00
Dams	0.50	0.80	0.45	0.65		
Drydocks	0.65	1.10	0.35	0.50		
Locks	0.65	1.10	0.35	0.50		
Girders	......	......	......	......	1.30	2.00
Slabs	......	......	......	......	2.80	3.50

The data as to costs and the estimates given in the succeeding pages must be taken as guides only and not absolute in any sense of the word. The method of arriving at the unit cost of doing any particular item of work will be explained, so that the estimator can apply the same process to the wages and conditions for his locality.

The cost of constructing earth coffer-dams or an embankment to exclude a small head of water is usually small in amount or nothing at all, the material from the excavation being banked up as it is dug out with the clam-shell or excavator, and where this cannot be done it will usually be found cheaper to use sheet-piling than to haul clay any distance. Where a log crib is necessary to prevent the clay from washing out, the small logs can be figured at a piling price per lineal foot and an equal amount will usually pay for the labor of placing them. The log crib coffer-dam on Green River, Fig. 21, for a distance of approximately 200 feet across the stream, cost about $2500, or about $12.50 per lineal foot of pipe. Counting cross cribs, there was about 500 feet constructed at approximately $5 per lineal foot, the excavation from a tunnel forming the filling at no extra cost.

The cost of sheet-pile coffer-dam labor comprises first the cost of making the piles. For Wakefield sheet-piling this will be usually about $3 per thousand feet, board measure, for boring, bolting, spiking and sharpening them. For dovetail tongue-and-groove piling, the dovetail strips must be figured at a greater cost than the ordinary

lumber, and the cost of boring these, spiking them on and sharpening the piles will run about $2.50 per thousand on the total lumber used.

The cost of driving both sheet-piles and round piles should never be taken at former unit costs in making up an estimate, without checking it up after the following manner.

Where a land driver is to be built, or one shipped in and assembled, or a floating driver towed in, this cost and the removal when the work is completed must be the initial charge against the work. Assuming there is a land driver to be built with 40-foot leads, an engine shipped in and out, and the driver removed when the work

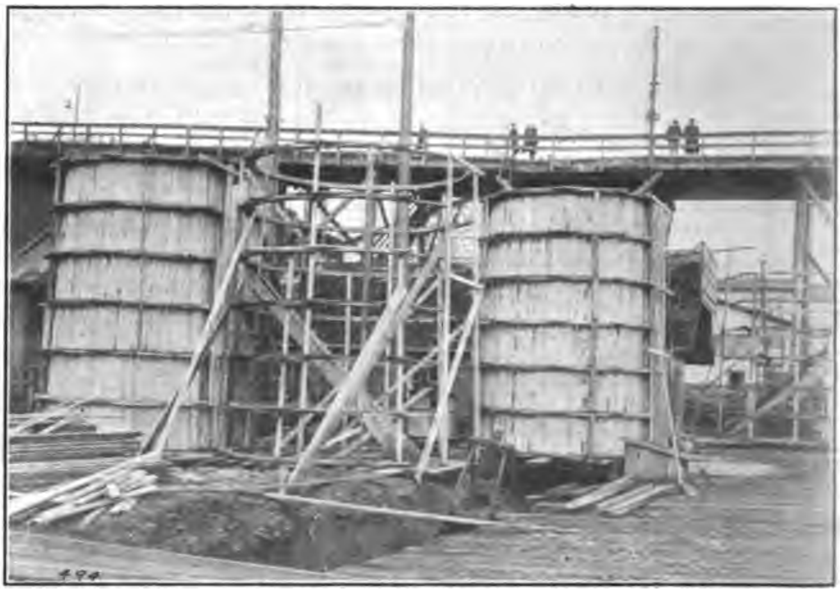

FIG. 461.—MAKING CURVED PIER FORMS ASHORE.

is done, then an item of cost must be added of approximately $250, or if there are 25,000 lineal feet of piling to be driven, one cent per lineal foot. If in the judgment of the engineer or estimator, 20 piles can be driven each day on the average, the cost per pile with crew, fuel, incidentals and repairs costing $30 per day will be $1.50 per pile, or with piles 50 feet long, 3 cents per lineal foot, to which must be added the one cent per lineal foot for plant, also the cost of cutting off arrived at by the same method, general expense and profit. This has been gone into in this elementary manner to illustrate the method to be employed in checking up all items of unit cost.

What is the total cost of the work per day divided by the average that can be accomplished each day, being sure that sufficient allowance is made in the estimate for lost time, repairs and the like in arriving at the average amount that can be done.

The cost of cutting off piles comprises the cost of material for scaffolds and usually the wages of only two men for each day. Where there is an extra amount of scaffolding to be done, four men may be required to keep up with the driver crew.

The average cost of driver crews per day will be about as follows:

Land Driver:		Scow Driver	
Foreman	$5.00	Foreman	$5.50
Crew (4)	14.00	Crew (5)	17.50
Engineer	4.00	Engineer	4.00
Fireman	3.00	Fireman	3.00
Coal	3.00	Boom man	3.50
Incidentals	1.00	Coal	4.50
Repairs	0.50	Incidentals	1.50
		Repairs	1.50
	$30.50		
Jetting extra:			$41.00
Crew (4)	14.00	Extra men heading (2)	7.00
Fireman	3.00		
Oiler	3.00		$48.00
Coal	6.00		
Incidentals	2.50	Cutting off crew (2)	7.00
Repairs	2.50		
	$31.00		

A scow driver crew on ordinary work should drive about 30 piles per day and on easy work, about 50 per day of eight hours. Where a jetting plant is used not more than one-fourth the number will be driven per day, and often as at Tacoma not more than two will be driven in an eight-hour day. The longest time on record per pile so far as the author knows was on the Buda-Pesth coffer-dam, described in Chapter I, or twelve to fourteen days per pile! This simply indicates that the work should not have been carried out by such methods.

The cost of operating a pumping plant for pumping out a coffer-dam will be about as follows for a 6-inch centrifugal; pump-man, $3.50; fireman, $3.00; coal, $4.00; incidentals, $1.00; and repairs, $1.50, or a total of $13 per day of eight hours. To which must be added the cost of installing and removal.

The cost of the floating pile-driver, described in Chapter IV, Figs. 44 to 48, inclusive, is given in the following itemized estimate:

UNITED STATES PILE DRIVER. Sept., 1912

24′×70′×4′7½″—66′ leads

Lumber—scow	26 M @ 30.00	$ 780.00
(Lbr. 15.50+Hand. 0.50+Labor 14.00)		
Lumber—deck	7 M @ 36.00	252.00
(Lbr. 25.00+Hand. 0.50+Labor 10.50)		
Leads	8 M @ 36.00	288.00
(Lbr. 15.00+Hand. 0.50+Labor 20.50)		
Loft deck in places	3 M @ 25.00	75.00
House	4½ M @ 35.00	157.50
Ship Knees		
8×8×3 ft.	19 @ 30.00	570.00
8×8×5 ft.	9 @ 45.00	405.00
Calking	6500′ @ 0.04	260.00
Hardware, bolts and spikes		309.00
Channels, bolts, etc.		108.00
House hardware		79.00
House canvas covering		35.00
Cleats, 42 in	4 @ 14.00	56.00
Sheaves	22	128.00
Guides and fittings in place		120.00
Hose 2½ in. double jacket	65′ @ 0.75	48.75
Painting		497.50
Pipes and valves		125.00
Boiler and pipe covering		168.00
Hammer	3800 lbs. @ 0.04	152.00
Oil-tanks	2 @ 126.00	252.00
Air-tank		135.00
Engine, 8¼×10		1,450.00
Boiler, 40 horse-power		820.00
Steam capstan		550.00
Duplex boiler feed-pump		33.50
Worthington jet-pump		390.00
Air-compressor		720.00
Tank, 50 gal.		25.00
Capstan		550.00
Ratchet gypsy windlass		28.00
Ratchet gypsy half windlass (2)		32.00
Feed-water heater		83.00
Installing machinery		775.00
Testing machinery		100.00
Delivery of driver		50.00
		$10,607.25
General Expense, 6 per cent		636.45
		$11,243.70
Profit, 10 per cent		1,124.40
		$12,368.10

This is undoubtedly a high cost, as the construction of the driver was let at practically $10,000. But it was evident that this bidder

added nothing for general expense and probably a very low percentage of profit. The cost could be still further reduced by omitting the ship knees and cutting out part of the painting, thus reducing the actual cost of the driver to below $9500.

The method of figuring the cost of deep coffer-dams is well shown by the Salmon Bay work described in Chapter VI, Fig. 91, where the depth was about a maximum for this class of construction. The original costs have been revised to take account of the increased expenditure due to the error in borings.

SALMON BAY PIERS, N. P. RY. Oct., 1912

Three (3) Coffer-dams

Excavation	4700 c. y. @ 1.75	$8,225.00
Sheet-piling:		
Lumber	180 M @ 12.00	2,160.00
Bolts	9000 lbs. @ 4.00	360.00
Framing	180 M @ 5.00	900.00
Driving sheet-piles	526 @ 2.00	1,052.00
Bracing:		
Lumber	75 M @ 12.00	900.00
Framing and placing	75 M @ 10.00	750.00
Pumping	50 days @ 26.00	1,300.00
		$15,647.00
Removing coffer-dams		1,036.00
General expense, 6 per cent		1,000.00
		$17,683.00
Profit, 12½ per cent		2,210.40
		$19,893.40
Each		6,631.13

Cement on work		
Cement f.o.b. cars	$1.85	
Handling	.10	
Haul	.20	
		$2.15 per bbl.

Sand and gravel:		
Bunker price	$0.35 per c.y.	
Tow, 40 miles	0.15	
Scow	0.15	
Launch tow, 1 mile	0.10	
Unloading	0.10	
Plant expense	0.05	
		0.90 per cu.yd.

Concrete 1-3-5:

Cement	1.17 bbls. @ 2.15	$2.52
Sand	0.5 c. y. @ 0.90	0.45
Gravel	0.9 c. y. @ 0.90	0.81
Mixing and placing		0.90
Forms and labor (old lumber)		0.50
Per cubic yard		5.18
General Expense, 6 per cent		.32
		5.50
Profit, 12½ per cent		.70
		$6.20

Concrete, 1-2-4:

Cement	1.5 bbls. @ 2.15	$3.22
Sand	0.5 c. y. @ 0.90	0.45
Gravel	0.9 c. y. @ 0.90	0.81
Mixing and placing		1.00
Forms and labor (old lumber)		0.50
Per cubic yard		5.98
General Expense, 6 per cent		0.36
		6.34
Profit, 12½ per cent		.79
		$7.13

FIG. 462.—CURVED FORMS ABOVE COFFER-DAM.

The excavation exceeded the rate given, as did also the pumping, but the estimate as made up gives the rates that would have governed had the borings been correct and the construction put in originally to correspond. This is a good illustration, however, of the need for using unit costs with caution, on account of the conditions being different from those assumed or given by the plans and specifications drawn to cover the work.

The following estimate is typical of the ordinary dredged piers as built for a common highway bridge in the Puget Sound country, although the borings did not prove to be correct and piles had to be driven in place of the rails.:

The cost of cement, sand, and gravel includes delivery at the work, while the labor item covers all handling on the work, as well as labor of mixing and placing concrete. The cost of forms was figured out in detail and an average used for the piers and another average value for the abutments. The seemingly high cost of the forms was due to all the work being medium thickness reinforced work. The item of $1.00 used for general expense was high owing to some plant expense being included.

ESTIMATE FOR TWO PIERS AND TWO ABUTMENTS

Two Piers:				
Footings 1–2–3 concrete:				
Cem. Sand Grav, Forms Labor Gen. Ex. Water				
3.40+0.52+0.77+1.80+3.00+1.00 + 0.11	60 cu. yds.	@	$10.60	$ 636.00
Shaft of pier 1–3–5– concrete:				
2.28+0.52+0.87+1.80+1.53+1.00 + 0.10	137 cu. yds.	@	8.10	1109.70
Coping of piers 1–2–3:				
3.40+0.52+0.77+1.80+1.51+1.00 + 0.10	33 cu. yds.	@	9.10	300.30
Foreman extra time..............................				200.00
Excavation heavy gravel..................	322 cu. yds.	@	4.00	1288.00
Sheeting..				400.00
Total concrete average....................	230 cu. yds.	@	17.10	$3934.00
Two abutments:				
Footings and body 1–3–5– concrete:				
2.28+0.52+0.87+1.20+2.03+1.00+0.10	50 cu. yds.	@	$8.00	$ 400.00
Coping 1–2–3–concrete:				
3.40+0.52+0.77+1.20+2.00+1.00+0.11	20 cu. yds.	@	9.00	180.00
Foreman extra time..............................				100.00
Excavation and backfill....................	140 cu. yds.	@	1.30	182.00
Total concrete, average....................	70 cu. yds.	@	$12.30	$ 862.00
Reinforcing bars:				
Steel Plac. Haul				
3.00+1.00+0.10..........................	1200 lbs.	@	$4.10	$49.20
Rails Haul Driver Driving				
Steel rails (for piles), 12.00+0.15+5.00+3.00.........	16	@	20.15	322.40
Total of all items....................................				$5167.60

The ordinary piers for a railway bridge in Oregon were estimated as follows, with general expense and profit to be added. The mass in the base consisted of timber cribs sunk by open dredging. The cutting edges were of reinforced concrete, and this was the cause of the high unit cost for the "mass in base," but without the reinforced concrete cutting edges, the cost was reduced over $5 per cubic yard as indicated.

The low cost of concrete was due largely to the fact that it was to be hauled near to the bridge site by the railway company. The entire estimate is only to be used as a guide making a similar estimate.

EIGHT ORDINARY DREDGED CAISSONS FOR RAILWAY PIERS

Mass in bases	2,680 cu. yds.				
Reinforced concrete. Cutting edge 1–3–5	944 yds.	@	$8.05	$7599.20	

Cement $1.65	Cement.... 1.17	@	1.95 = 2.28		
¼ Frt.......... .05	Sand....... .5	@	1.00 = .50		
¼ Haul......... .15	Gravel..... .82	@	1.00 = .82		
Handling....... .10	Forms.... 29.20	@	.08 = 2.45		
	Water......		.10 = .10		
$1.95	Labor mix. an pl.		1.90 = 1.90	8.05	

Reinforcing steel	202,000 lbs.	@	2.70	5,454.00	

Frt..... .10
Haul.... .30
4) .40(.10

Steel 2.00 + Frt. and Hl. .10 + Plac. .60

Cutting edges	58,216 lbs.	@	0.08	4,657.28	
Plates	19,040 lbs.	@	0.07	1,332.80	
Crib lumber	320 M	@	25.00	8,000.00	

Lbr. 12.00 + Frt. .66 + Haul. 1.50 + Plac. 10.84

Excavation	5,600 yds.	@	2.75	15,400.00	
Concrete in Seal 1–2–4	1,552 yds.	@	5.28	8,194.56	

Cement....	1.5 bbls.	@	1.95	2.92	
Sand........	.42 yd.	@	1.00	.42	
Grav........	.84 yd.	@	1.00	.84	
Water.......				.10	
Labor mix. and pl.				1.00	5.28

Coffer-dam lumber	217 M.	@	22.00	4,774.00	
Falsework, Piles, 288 @ 30 ft........	8,640 ft.	@	0.10	864.00	
Driving..................	288	@	5.00	1,440.00	
Lumber.................	30 M.	@	25.00	750.00	$58,465.84

With concrete cutting edge, 58,465.84 ÷ 2680 = 21.81 cu. yd.
Without concrete cutting edge = 16.66 cu. yd.

8 *Concrete piers* 1–3–5................					
Concrete shafts:	2,568 yds.	@	6.22	15,972.96	
Cement.... 1.17 @ 1.75 2.05					
Sand....... .5 @ 1.00 .50					
Grav...... .82 @ 1.00 .82					
Water.................... .05					
Labor mix. and plac...... 2.00					
Forms 6.7 sq. ft. @ .12 .80					
6.22					
Steel in nose......................	12,800 lbs.	@	0.05	640.00	
Reinforcing........................	4,680 lbs.	@	.027	126.36	16,739.32
16,739.32÷2568=$6.52 per cu. yd.				$16,739.32	

The method of arriving at the cost of pneumatic bases for the piers of an ordinary railway bridge to a depth of 40 feet below water is given in the following estimate. From this it will be seen that the principal items of cost are the excavation, sinking, and concrete. Under favorable conditions they could be very much reduced, but the judgment of the engineer and estimator must govern as to how low the item of excavation and sinking can be reduced. The item for falsework around cribs (Fig. 460) is often overlooked and plant must often be figured into the cost.

PNEUMATIC CAISSONS FOR RAILWAY BRIDGE PIERS

Air sinking:					
Mass in base.................	3,640 yds.				
Concrete 1–2–4....................	3,000 yds.	@	$7.38	22,140.00	
Cement.....1.5 @ $2.21 3.32					
Sand....... .42 @ 2.83 1.19					
Gravel..... .84 @ 1.00 .84					
Labor mix. and plac...... 2.00					
Water .... @ .03 .03					
$7.38					
Crib lumber:					
Lmbr. Frt. Haul. Plac.					
12.00 + 1.65 + 1.00 + 15.00	270 M.	@	29.65	8,005.50	
Cutting edge......................	35,340 lbs.	@	0.08	2,827.20	
Shafts 3 ft. and 2 ft. diameter......	54,000 lbs.	@	0.10	5,400.00	
Rods and bolts....................	20,000 lbs.	@	0.06	1,200.00	
Top coffer-dam...................	50 M.	@	22.00	1,100.00	
Excavation and sinking............	4,000 yds.	@	3.00	12,000.00	
False work:					
Piles, 144 @ 30 ft............	4,320 ft.	@	0.10	432.00	
Pile driving.................	144	@	5.00	720.00	
Lumber.....................	16 M.	@	25.00	400.00	$54,224.70

54,224.70÷3640=14.90 per cu. yds.+gen. exp. 0.90+prof. 2.40=$18.20 per cubic yard.

FIG. 463.—ANCHORING DREDGED CRIB AT TACOMA.

DREDGED CAISSONS FOR TACOMA BRIDGE

The labor costs on the Tacoma dredged cribs and concrete work described in Chapter X are based on common labor at $2.50 per eight-hour day, carpenters $3.50 per day, pile-driver men $3.50 per day, caisson men $4.00 per day and engineers $4.50 per day.

The abutment while of reinforced design (Fig. 464) was quite heavy, except the cantilevers. The labor on forms was $0.83 per cubic yard and labor mixing and placing concrete $1.83 per cubic yard.

The pedestals for the viaduct columns were quite heavy, about 15×15 feet on the base and were all in the dry. The labor on the excavation was $1.20 per cubic yard including backfill. The digging was on the average, of medium hardness; the labor on forms was $0.26 per cubic yard and the labor mixing and placing concrete $0.92 per cubic yard.

The labor on the cribs and on the mass in the base of the piers was for the timber framing, $9.65 per thousand without the cost of rigging up yards and ways and removing same but with these included was $18.75 per thousand; the average cost of sinking on piers Nos. 1, 2, 3 and 4 was $2.42 per cubic yard but leaving out No. 1 which was in hard material, the cost was $1.81 per cubic yard; the labor of mixing and placing concrete was $1.48 per cubic yard; and the labor on driving piles was $0.26 per lineal foot below cutting edge.

The pier shafts had high labor costs owing to their being of reinforced concrete and owing to the expensive forms. (Figs. 461 and 462.) The labor on forms was $1.61 and on mixing and placing concrete $1.19 per cubic yard.

The labor was much higher than in other parts of the world and for this reason not so efficient as a matter of course.

The piers were very high and the cost of placing concrete was much above the average. Probably considerable saving would have resulted from using a tower on a scow from which to spout the concrete, although with an 18-foot rise and fall of tide every day, it would have been difficult to handle.

PNEUMATIC CAISSONS, VANCOUVER BRIDGE

The costs of the work on the Vancouver, Wash., bridge were the result of ideal conditions as to weather, material through which to sink, and as to cost of lumber. When the work was begun on this treatise it was the intention to give these costs, but mature consider-

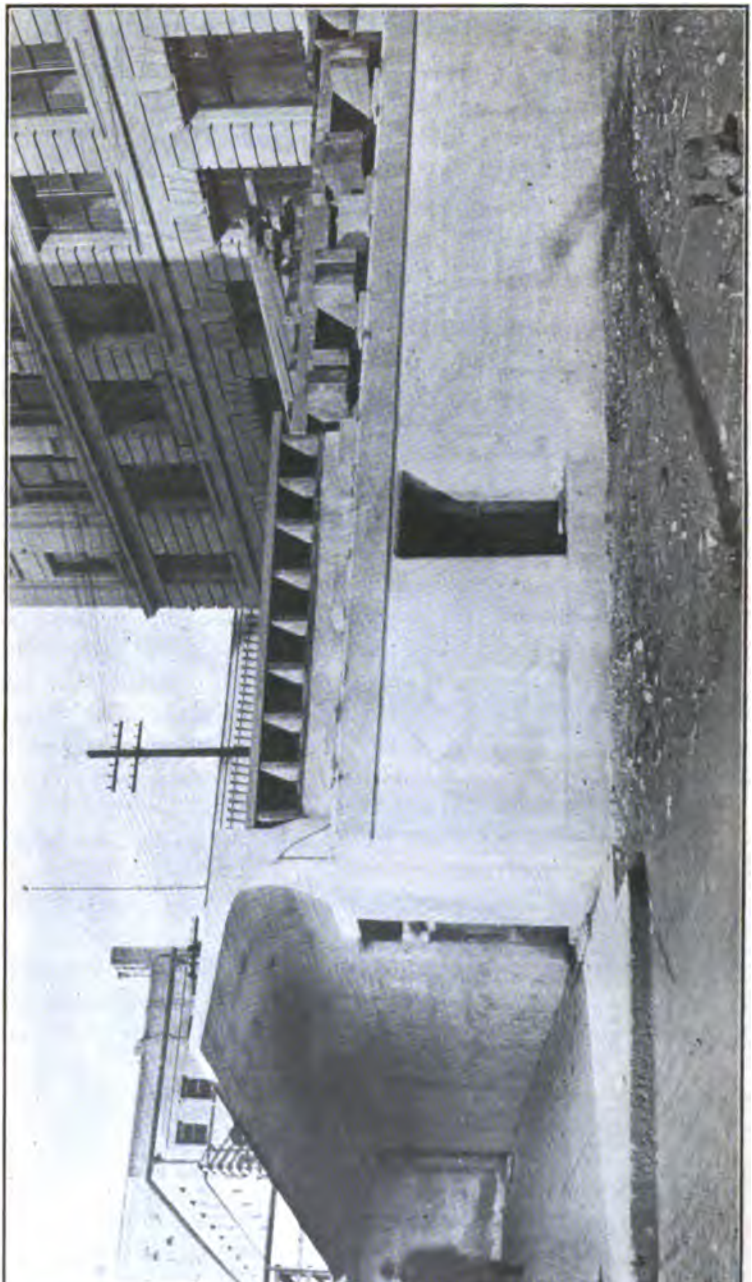

FIG. 464.—REINFORCED CONCRETE ABUTMENT, TACOMA.

ation leads the author to conclude that they would be misleading in the extreme and for that reason they will be given on the basis of ordinary conditions about as represented by caisson No. 3.

The labor of building the caissons includes the cost of launching, towing to position, driving dolphins to hold in place till landed on bottom, and building up as caissons were sunk, and the wages of general foreman.

The labor of sinking includes all the pressure men or sand hogs, outside lock tenders and watchmen, general foreman and all force account work on Power Barge, including a proportion of wages of master mechanic; also includes the cost of placing the concrete inside of working chamber that was done by sand hogs.

The labor of concreting includes all cost of handling material, namely, cement into and out of cement house, towing sand and gravel from digger, etc., also the proper proportion of the wages of the general foreman. The cost of cement includes $1.71 per barrel for freight.

Caisson pier No. III:

Labor per M. B. M. on timber	18.10
Timber per M. B. M.	11.10
Rods, bolts, spikes and oakum per M. B. M	5.43
Locks and shafts per M. B. M	7.60
Miscellaneous per M. B. M..	.67
Gross volume including working chamber 68,826 cu. ft.	
Labor per cu. ft	0.0741
Timber per cu. ft	0.0456
Iron and oakum per cu. ft	0.0223
Locks and shafts per cu. ft	0.0312
Miscellaneous per cu. ft	0.0028
Total per cu. ft. gross volume	0.176

Concrete pier III:

Labor per cu. yd	0.638
Cement per cu. yd	3.264
Sand and Gravel per cu. yd	0.722
Miscellaneous per cu. yd	0.213
Total per cu. yd	4.838

Sinking pier III:

Labor per cu. ft. of gross volume	0.1584
Fuel and miscellaneous per cu. ft. of gross volume	0.0317
Total cost per cu. ft. of gross volume	0.1900
Labor per cu. ft. sunk below low water	0.1241
Fuel and miscellaneous per cu. ft. sunk below low water	0.0248
Total cost per cu. ft. sunk below low water	0.1487

Pier III was expensive on account of being the first and being charged with several delays waiting for material, also on account

of going 5 feet into boulders and cemented gravel which had to be blasted and hoisted out. All material in the other caissons was removed by the wet blowout process, no sand pump or ejector being used.

The making of the detailed estimate of cost for a pier or wharf is not very difficult but to be sure of the character of the bottom, the conditions surrounding the work, and every thing that will affect the cost is the all important matter.

The keeping of costs to be of any value must be on a uniform basis from year to year, and it has been the author's custom to have the cost of framing and placing lumber, and the cost of driving piles include all plant installation, rigging and incidentals, which will explain the seemingly high unit costs in the following estimates of piers and wharves.

The ordinary type of pier with creosoted piles is well shown by that at Fort Ward on Puget Sound, and the method of estimating the cost can be seen from the following estimate from which the structure was actually built. The houses on the wharf were very small and the cost was figured at a seemingly high rate per cubic foot. The driving of the piles could be sub-let to a small concern well inside the cost given and the same is true regarding the framing and placing of the lumber.

However if every contractor would figure on the basis shown, there would be a much better tone to the construction business.

FORT WARD CREOSOTED PILE PIER

$20 \times 328 + 70 \times 100 = 13{,}560$ Sq. Ft.

Pier (not including buildings):

Bearing piles creosoted	7350 ft.	@	$0.36	$2646.00
Piles @ 0.35 and towing 0.01.				
Driving piles and capping	207	@	3.00	621.00
Fender piles creosoted	2250 ft.	@	0.39	877.00
Piles 0.38, towing 0.01.				
Driving piles	40	@	3.00	120.00
Bracing lumber creosoted	24 M.	@	46.00	1104.00
Lumber 38.00, towing 3.00, placing 5.00.				
Lumber	119 M.	@	11.50	1368.00
Towing	119 M.	@	1.00	119.00
Framing and placing	101 M.	@	4.00	404.00
Logs in float in place	3 M.	@	10.00	30.00
Bitts	6	@	15.00	90.00
Placing bitts (2 old)	8	@	2.00	16.00

Hardware:					
Bolts	3300 lbs.	@	$6.00	$198.00	
Drift bolts	2800 lbs.	@	2.50	70.00	
Washers	2700 lbs.	@	2.60	70.00	
Boat spikes	1600 lbs.	@	3.00	48.00	
Nails and spikes	2000 lbs.	@	3.00	60.00	
Galvanized wire rope	300 ft.	@	0.10	30.00	
Galvanized pipe 1 inch	60 ft.	@	0.12	7.00	
Sill covering	5000 sq. ft	@	0.03	150.00	
Tearing out old wharf				550.00	
Grading approach				100.00	
Relaying track				10.00	
					8688.00
General Expense, 6 per cent					521.30
					$9209.30
Profit, 12½ per cent					1151.15
Cost per square foot				$0.76	$10360.45

Store house 10×40×8 ft.=3200 cu. ft.		@	$0.15	480.00	
Boat house No. 1 16×40×8=5120 cu. ft.		@	0.08	408.00	
Boat house No. 2, 52×24×8=9984 cu. ft.		@	0.08	798.00	
Piles in boat houses, 830 ft.		@	0.38	315.00	
Driving 24 piles		@	3.00	72.00	$2073.00
General Expense, 6 per cent					124.40
					$2197.40
Profit, 12½ per cent					274.60
					$2472.00

The estimate for the Tacoma Smelting Co. pier is for a heavy wharf to carry piles of ore and is very nearly as heavy as those built by the navy department to carry light trains, guns, and warship material.

TACOMA SMELTING CO. PIER, TACOMA, WASH.

540×40=21,600 Sq.Ft.

Piles creosoted	34,925 ft.	@	$0.50	$17,462.50
Towing piles	34,925 ft.	@	0.005	174.60
Driving piles	635	@	2.50	1,587.50
Painting piles	635	@	0.50	317.50
Lumber merchantable	107 M.	@	10.00	1,070.00
Lumber—deck	86 M.	@	13.00	1,118.00
Framing and placing	193 M.	@	5.00	965.00

Caps	44	@	$10.00	440.00	Average $5.00 per M.
Stringers	62	@	3.50	217.00	
Sheeting	22	@	5.00	111.00	
Plank	65	@	3.00	195.00	
	193	@	4.98	962.00	

Towing lumber 3 scows	2 trips	@	$50.00	$100.00

Wrought iron:

Drift bolts	1650 lbs.	@	0.04	66.00
8-in. spikes wire	2900 lbs.	@	0.03	87.00
40 d. nails	500 lbs.	@	0.03	15.00
10 d. nails	500 lbs.	@	0.03	15.00
				$22,978.10
General Expense, 6 per cent				1378.70
				$24,356.80
Profit, 12½ per cent				3,044.60
				$27,401.40
Cost per square foot				$ 1.27

The most recent timber pier for the navy yard on Puget Sound (Fig. 465) described in Chapter XXVII, with the piles and bents 8 feet center to center, is of the heaviest type of wooden pier ever built.

The estimate of cost gives all the details entering into the construction.

PUGET SOUND NAVY YARD, PIER NO. 5

80×504 FT. 5 IN. = 40,350 SQ. FT.

Piling (creosoted) 16-in. butt and 9-in. top	53,758 ft.	@	$0.40	$21,503.20
Piling (creosoted) 20-in. butt and 9-in. top	1,452 ft.	@	0.42	609.84
Piling, untreated	9,166 ft.	@	0.12	1,099.92
Towing	64,376 ft.	@	0.005	321.88
Driving straight piles	701	@	2.50	1,752.50
Driving bollard piles	22	@	4.00	88.00
Driving brace piles	244	@	6.00	1,464.00
Driving corner brace piles	8	@	6.00	48.00
Driving fender piles	143	@	3.00	429.00
Pile cut-off	1,116	@	0.50	558.00
Pile points	1,116	@	1.00	1,116.00
Pile Notching:				
Bearing piles	690	@	0.50	345.00
Brace piles	252	@	0.50	126.00
Lumber	442 M.	@	19.70	8,707.00
Lumber, towing	12 scows	@	30.00	360.00
Lumber, framing and placing	442 M.	@	5.00	2,210.00
Wrought iron:				
Screw Bolts (galvanized)	25,385 lbs.	@	5.35	1,358.10
"U" bolts (galvanized)	6,410 lbs.	@	5.35	342.94
Drift bolts (galvanized)	10,730 lbs.	@	5.35	574.06
Angles bolts (galvanized)	3,630 lbs.	@	5.35	194.20
Spikes boat (galvanized)	6.5 tons, 62 kegs	@	3.60	223.20

FIG. 465.—DECK OF NAVY YARD PIER.

Rails	27 tons	@ 40.00	1,080.00	
Rail joints	76 tons	@ 3.00	228.00	
Spikes, railway	600 lbs.	@ 0.10	60.00	
Bolts	400 lbs.	@ 0.25	100.00	
Frog	1	@ 50.00	50.00	
Switch	1	@ 50.00	50.00	
Cast iron:				
Washers (galvanized)	17,753 lbs.	@ 5.50	976.42	
Bollard caps	4,000 lbs.	@ 4.00	160.00	
Towing iron	66 tons		150.00	
Laying track	2,260 ft. rail		100.00	
Carbolineum	3 bbls.	@ 30.00	90.00	
White lead	10 gals.	@ 1.00	10.00	
Pitch	20 gals.	@ 1.00	20.00	
				$46,505.26
General expense, 6 per cent				2,790.30
				$49,295.56
Profit, 12½ per cent				6,161.95
				$55,457.51
Cost per square foot				1.37

The cost of concrete piles for a piece of work with only 3000 linear feet of piles to be driven is given below, but the cost of concrete could be very greatly reduced as it includes the cost of forms; and the cost of driving could be reduced very greatly with a larger number of piles to be driven.

COST CONCRETE PILES PER LINEAR FOOT

Concrete	$0.490
Bars and wire	0.214
Driving	0.606
Tests	0.031
Driver	0.092
	$1.433
General expense	0.087
Cost	$1.520

The reinforced concrete railway dock at Vancouver, B. C., was similar in many respects to the one described in Chapter XXVII as built at the navy yard on Puget Sound, but many changes had to be made during the construction to meet conditions that had not been forseen, and the estimate is given simply as a guide to the making of one for a similar structure. The cost of piles in the first item is for falsework piles to support the forms.

CONCRETE-STEEL RAILWAY DOCK, VANCOUVER, B. C.

450×400 Ft. = 180,000 Sq. Ft.

				Labor and Material.
Floor System Concrete (1 : 2 : 4)................	4190 yds.	@	$12.16	$50,950.40

C. S. G. W. M. & P. F. Piles
3.06+.66+1.34+.10+1.50+3.00+2.50

Wall and land piers (1 : 2 : 4)..................	260 yds.	@	10.16	2,641.60

C. S. G. W. M. & P. F.
3.06+.66+1.34+.10+1.50+3.50

Transverse ties (1 : 2 : 4)......................	65 yds.	@	20.16	1,310.40

C. S. G. W. M. & P. F. Excav.
3.06+.66+1.34+.10+2.00+3.00+10.00

Cylinder piers:

Concrete footings (1 : 2 : 3)....................	940 yds.	@	18.52	17,408.80

C. S. G. W. M. & P. Dredg.
3.60+.80+1.20+.10+ 6.00 + 6.82

Concrete shell (1 : 1½ : 2) (1406) *lin. ft.*)	184 yds.	@	18.12	3,334.08

C. S. G. W. M. & P. F. Break.
4.60+.72+1.20+.10+4.00+6.50+1.00

Erecting shells..............................	38	@	120.00	4,560.00

Weight of each shell 10 tons
Erection 10 tons @ 12.00=120.00

Reinforcing in shell and filler..................	97,660	@	.0335	3,270.61

Material............... 97,660 lbs. @ .0235
Labor................. 97,660 lbs. @ .01

Concrete filler (1 : 2 : 3)......................	655 yds.	@	7.70	5,043.00

C. S. G. W. M. & P.
3.60+.80+1.20+.10 + 2.00

Concrete piles (1 : 2 : 3½)......................	6788 lin. ft	@	2.20	14,933.60

16×16 ins.×46 ft. long
C. S. G. W. M. & P. F.
3.32+.72+1.25+.10+2.50+4.00=11.89

Royalty.............................			.10
			.03
Concrete 1.8 cu. ft.	labor......	.245 =	.44
	material....	.196 =	.35
Bars, 28.5 lbs.	labor.......		.20
	material....		.67
Driving, .17+bracing, .10			.27
Breakage (6×46 ft. ×$1.70)÷6788.......			.07
Driver erection........................			.07
			2.20

Excavation dry fill..........................	350 yds.	@	1.00	350.00

Reinforcing steel other than in					
Piles and piers cylinders......................		376 tons	@	60.00	22,560.00
Material, .0235×2000 lbs.=47.00					
Labor, .0065×2000 lbs.=13.00					
Castings, etc..................................		8 tons	@	100.00	800.00
Labor, 20.00+material, 80.00					
Bolts, 1258....................................		2 tons	@	140.00	280.00
Labor, 60 00+material, 80.00					
Coffer-dams (10×10×25 ft. high).............		38	@	155.00	5,890.00
Wakefield sheet piling, 40 piles @ 12 ft. long					
Lumber in each 112-ft. B.M. @ 14.00=	1.57				
Framing in each B.M. @ 4.00	.45				
Driving each pile....................	3.00				
	5 02				
40 Piles @ 5.00.	200 00				
Bracing...........................	25.00				
Average, 155.00 each.	225.00				
Diver, 4 months............................			@	400.00	1,600.00
Duty......................................					1,000.00
General Expense, 6 per cent .... 8,155.80........................					$135,932.49
Profit, 12½ per cent .... 18,011.10........................					$144,088.29
					$162,099.39

CHAPTER XXXIII

ESTIMATING THE COST—*Continued*

The design of pier No. 8 at the Puget Sound navy yard is much lighter than the later ones to be constructed and it represents a type suitable for a commercial wharf where it is desired to construct a moderately heavy reinforced concrete structure. The design is similar to that for pier No. 4 described in Chapter XXVII. The cost per square foot is $3.30, or nearly the same as those recently built at San Francisco.

U. S. GOVERNMENT PIER NO. 8, PUGET SOUND NAVY YARD

REINFORCED CONCRETE. 60×403 FT. = 24,180

Concrete above piles (1-2-4)........	1,100 cu. yd.	@	$13.25	$14,575.00
Cement....... 1.5 @ 2.40 = 3.60				
Sand......... 0.5 @ 1.10 = .55				
Gravel....... 1.00 @ 1.10 = 1.10				
Labor...................... 2.00				
Forms, 70,000 sq. ft. surface				
= 175 M. @ 14.00 = 2350				
Labor @ 25.00 = 4375				
6725 = 6.00				
13.25				
Granolithic, 25,000 sq. ft.	90 cu. yds.	@	20.74	1,867.00
Cement.... @ 2.40×3.21 = 7.70				
Sand...... @ 1.10× .95 = 1.04				
Labor.................... 12.00				
20.74				
Expanded metal..................	3,900 sq. ft.	@	0.08	312.00
0.04+0.04				
Reinforcing bars..................	116,000 lbs.	@	3.20	3,712.00
2.50+0.10+0.60				
Fender piles......................	3,400 ft.	@	0.20	680.00
0.10+0.10				
Galvanized U-bolts, washers, etc.....	3,000 lbs.	@	0.15	450.00
Fender wales......................	7 M.	@	40.00	280.00

Slip lumber	5 M.	@	25.00	125.00	
Chain	1,000 lbs.	@	0.10	100.00	
Sheaves brackets				50.00	
2 Counter weights	1,000 lbs.	@	4.00	400.00	
Wild cats	2	@	500.00	1,000.00	
Rails	42,000 lbs.				
Structural steel	332,000 lbs.				
1.97+0+80.05+1.00	374,000 lbs.	@	3.82	14,287.00	
Sway rods	17,000 lbs.	@	10.00	1,700.00	
Hemp buffers	2	@	25.00	50.00	
Hy. rib	30,000 sq. ft	@	7.00	2,100.00	
5.90+1.10					
Reinforcing bars	28,000 lbs.	@	3.60	1,008.00	
250.+0.10+1.00					
Cutting edge	3,000 lbs.	@	20.00	600.00	
Forms average surface 300 sq. ft. per shell					
300 sq. ft.×0.07=21.00×100	100	@	21.00	2,100.00	
Erecting shells	100	@	60.00	6,000.00	
Average weight 5.5 Ts per shell					
550 Ts. @ 12.00					
100 Piles. @ 60.00					
Concrete in shells=1-2	270 yds.	@	20.00	5,400.00	
Cement 2.40×3.21=7.70					
Sand 1.10× .95=1.04					
Labor 11.26					
20.00					
Concrete filler=1-2-4	700 yds.	@	7.25	5,075.00	
Material 5.25					
Labor 2.00					
14 Cleats	3,000 lbs.	@	.10	300.00	
Cutting old wall				250.00	
Rentals				500.00	
Plant 1,000.00, tracks 500.00, houses 800.00				2,300.00	
Interest 700.00, bond 375.00				1,075.00	
Demurrage	60 days	@	10.00	600.00	
					$66,896.00
General Expense, 6 per cent					4,013.75
					$70,909.75
Profit, 12½ per cent					8,863.72
					$79,773.47
Cost per square foot					3.30

The heavy reinforced concrete pier constructed at the Puget Sound navy yard in 1913, was described in Chapter XXVII and illustrated in Figs. 380, 381 and 382. The contract was let slightly under the cost shown in the following estimate, but at figures which were manifestly too low, if any part of the general expense was to be earned, to say nothing of any profit.

The pile approach covers a distance that later on will be an earth fill, inside a new sea wall at the inner or shore end of the concrete wharf.

U. S. GOVERNMENT PIER NO. 4, PUGET SOUND NAVY YARD

CONCRETE WHARF 490×80 FT. = 39,200 SQ. FT.

PILE APPROACH 210×50 FT. = 10,500 SQ. FT.

Concrete:			
Cylinders, shells 68 @ 12 yds...........	825 cu. yds.	@ $18.82	$15,526.50
Proportion 1 cement to 4¼ sand and gravel			
Cement.......... 2.00 @ 2.10 = 4.20			
Sand............ .42 @ .85 = .36			
Gravel.......... .84 @ .85 = .72			
Forms 118 sq. ft.. @ .08 = 9.44			
Mixing and placing............ 3.00			
Water........................ 0.10			
Plant........................ 1.00			
18.82			
Placing shells...........................	68	@ 50.00	3,400.00
Gov. derrick 8 hrs., $48.00 ÷ 4 = 12.00			
Labor bracing, etc........ 38.00			
Cylinders filling 68 cyl. @ 32 yds..........	2,125 cu. yds.	@ 5.96	12,665.00
Proportion 1 cement, 6½ sand and gravel			
Cement.......... 1.41 @ 2.10 = 2.96			
Sand............ .40 @ .85 = .34			
Gravel.......... .89 @ .85 = .76			
Forms......................... 00			
Mixing and placing............... .80			
Water........................ .10			
General.......................1.00			
5.96			
Concrete caps 1 : 1½ : 3..................	723 cu. yds.	@ 13.73	9,926.80
C. S. G. F. M. & P. W. Plt.			
4.20 + .36 + .72 + 5.85 + 1.50 + .10 + 1.00			
Columns over cylinder 1 : 1½ : 3..........	442 cu. yds.	@ 11.18	4941.55
C. S. G. F. M. & P. W. Plt.			
4.20 + .36 + .72 + 3.30 + 1.50 + .10 + 1.00			
Floor slab, 1 : 1½ : 3....................	1,075 cu. yds.	@ 11.63	12,502.25
C. S. G. F. M. & P. W. Plt.			
4.20 + .36 + .72 + 3.75 + 1.50 + .10 + 1.00			
Girder casing, 1 : 1½ : 3..................	706 cu. yds.	@ 15.83	11,176.00
C. S. G. F. M. & P. W. Plt.			
4.20 + .36 + .72 + 7.95 + 1.50 + .10 + 1.00			
Reinforcing steel:			
¾ in. and over.........................	318,350 lbs.	@ 2.55	8,117.90
⅝ in...................................	183,850 lbs.	@ 2.60	4,780.10
⅜ in...................................	8,390 lbs.	@ 2.80	234.95
	510,590 lbs.		
Freight, Seattle to Navy Yard............	256 tons	@ 1.50	384.00
Loading 1.00 and tow..................	3 scows	@ 40.00	120.00
Placing in shells.......................	37 tons	@ 12.00	444.00

Placing in cylinder filling	78 tons	@	9.00	702.00
Placing in caps	82 tons	@	10.00	820.00
Placing around girders	5 tons	@	10.00	50.00
Placing floor slab	56 tons	@	10.00	560.00
Electric fabric 4×4×8 wire Material 1.75, labor 2.00	56,000 sq. ft.	@	3.75	2,100.00
No. 28 gauge, 4 rib, hy.-rib Material 4.25, labor 2.00	43,350 sq. ft.	@	6.25	2,709.30
Angles in caps 2–6×4×½ in.	48,600 lbs.	@	5.00	2,430.00
Structural steel	1,164,400 lbs.	@	4.00	46,576.00
Freight, Seattle to yard	583 tons	@	1.50	875.00
Erection	583 tons	@	10.00	5,830.00
Cleats C. I. bollards C. I. rail stops Manhole frame and cover Fire plugs	40,000 lbs.	@	4.00	1,600.00
Waterproofing for cement Material 0.11, labor 0.01	13,600 lbs.	@	0.12	1,632.00
Rails, 70 lbs. A.S.C.E., 2700 ft	32 tons	@	36.60	1,171.20
Continuous rail joints	82 tons	@	2.50	205.00
Frogs and switches	1 set	@	70.00	70.00
Laying track				600.00
Piling:				
Creosoted, 191 @ 57 ft	10,890 ft.	@	0.40	4,356.00
Driving	191	@	2.50	477.50
Cut-off	191	@	0.50	95.50
Pointing	191	@	0.50	95.50
Brace piles (creosoted) 50 @ 61 ft	3,050 ft.	@	0.40	1,220.00
Driving	50	@	3.50	175.00
Cut-off	50	@	0.75	37.50
Pointing	50	@	0.50	25.00
Bollards (creosoted) 4 @ 65 ft	260 ft.	@	0.50	130.00
Driving	4	@	5.00	20.00
Cut-off, pointing, rounding off top	4	@	1.50	6.00
Fender piles:				
Approach and pier, 193 @ 64 ft	12,310 ft.	@	0.10	1,231.00
Driving	193	@	2.50	482.50
Cut-off	193	@	0.50	96.50
Foundation piles, 702 @ 74 ft	5,1950 ft.	@	0.10	5,195.00
Driving	702	@	5.00	3,510.00
Cut-off	702 .	@	1.00	702.00
Lumber deck	34 M.	@	21.00	714.00
Labor	34 M.	@	4.00	136.00
Lumber, general	82 M.	@	13.00	1,066.00
Labor	82 M.	@	6.00	482.00
Iron:				
Drift bolts (galvanized)	5,000 lbs.	@	4.10	205.00
Machine bolts (galvanized)	6,900 lbs.	@	5.35	369.15
Exp. bolts (galvanized)	1,450 lbs.	@	5.50	79.75
U-bolts (galvanized)	5,200 lbs.	@	5.35	278.20
Angles (galvanized)	550 lbs.	@	5.50	30.25
Washers	900 lbs.	@	5.00	45.00
Boats spikes	900 lbs.	@	4.00	36.00
White lead	10 gals.	@	2.00	20.00
Carbolineum	2 bbls.	@	30.00	60.00

Cable for cluster piles	200 ft.	@	0.25	50.00
4-C. I. pipe	3,200 lbs.	@	5.00	160.00
Cluster piles, 12 @ 83 ft	1,000 ft.	@	.10	100.0
Driving	12	@	5.00	60.00
Cut-off	12	@	0.50	6.00
Wrapping				12.00
				$173,803.90
General Expense, 6 per cent				10,428.20
				$184,232.10
Profit, 12½ per cent				23,029.00
				$207,261.10
Cost per square foot concrete portion				4.50

The estimate of the concrete-steel viaduct described in its several features in the preceding pages and illustrated in Fig. 64 is given below in its revised form.

EAST TWENTY-FIRST STREET VIADUCT, PORTLAND

Concrete:				
Floor system 1-2-4	1,633 yds.	@	$14.11	$23,041.63
C. S. G. F. S. M. & P. W. Plt.				
2.80+0.50+0.90+4.67+2.72+0.94+0.08+1.50				
Columns 1-2-4	407 yds.	@	14.11	5,742.77
Abutments 1-3-5	958 yds.	@	6.41	6,140.78
1.92+0.50+0.90+1.02+0.00+0.52+0.05+1.50				
Footings 1-2-4	609 yds.	@	7.32	4,457.88
2.80+0.50+0.90+1.02+0.00+0.52+0.08+1.50				
Cement railing	695 ft.	@	2.21	1,535.95
Per Lin. Ft. Cem. S. & G. Forms M. & P.				
0.30 + 0.12 + 1.41 + 0.38				
Reinforcing steel	329 yds.	@	47.90	15,759.10
St. 37.00+unload 0.55+bend 2.16+placing 8.19				
Concrete piles	2,720 ft.	@	1.43	388.96
Concrete 0.49+steel 0.21+driv. 0.61+tests 0.03+driver 0.09				
Electric welded fabric	20,000 sq. ft.	@	0.03	600.00
Dry earth excavation	2,599 yds.	@	1.00	2,599.00
Wet earth excavation	753 yds.	@	4.00	3.012.00
Earth fill	5,800 yds.	@	0.33	1,914.86
Rock excavation	14 yds.	@	5.00	70.00
Lamp posts	14	@	45.00	630.00
Bitulithic pavement	4,283 sq. yds.	@	1.64	7,024.12
Cement sidewalk	5,814 sq. ft.	@	0.08	465.12
Concrete curb	119 lin. ft.	@	0.32	38.08
Plant not distributed				1,751.80
				$75,472.05
General Expense, 6 per cent				4,528.32
				$80,000.37
Profit, 12½ per cent				10,000.03
				$90,000.40

The estimate of three 60-foot reinforced-concrete arch spans which follows, gives the relative amounts which enter into the foundations and the substructure of such a bridge.

The costs of concrete, of steel and of the coffer-dams and erection are extremely low and yet the contract was let at figures below the cost given. When contractors learn to make each piece of work carry itself, then will the continual friction between engineers and contractors cease, as no contractor who has a losing job, or a very low priced one, will long remain in a calm state of mind unless he is receiving undue consideration from the engineer.

THREE 60-FOOT REINFORCED CONCRETE ARCHES, SPOKANE, WASH.

B or *C* concrete 1–3–5:				
Concrete	1275 yds.	@	6.20	$7,905.00
" *A* " concrete 1–2–4 arch ring:				
Concrete	1125 yds.	@	7.70	8,662.50
" *A* " Concrete 1–2–4 parapet and spandrel walls:				
Concrete	280 yds.	@	9.70	2,716.00
Mortar 1–3	75 yds.	@	1.00	825.00
Steel reinforcing:				
½ in., 13,400 lbs.				
⅝ in., 12,800 lbs.				
¾ in., 108,000 lbs.	174,200 lbs.	@	1.90	3,325.00
1 in., 40,000 lbs.				
Haul and erection: 2.00 and 23.00	174,200 lbs.	@	25.00	1,750.00
Excavation, dry	500 yds.	@	0.50	250.00
Excavation, wet and pumped	1000 yds.	@	1.50	1,500.00
Piling 25 ft	7,800 ft.	@	0.25	1,950.00
Pile driving	312	@	2.00	624.00
Pile driver				250.00
Lumber in bulkhead	2 M.	@	35.00	70.00
Fill	6,500 yds.	@	0.30	1,950.00
Grading	4,500 yds.	@	0.30	1,350.00
Water proofing	1,600 sq. yds.	@	0 30	480.00
				$33,603.50
Asphalt roadway	1,050 sq. yds.	@	2.00	2,100.00
Concrete sidewalks	450 sq. yds.	@	1.26	570.00
Electric lights complete	8	@	75.00	600.00
Riprap	175 yds.	@	1.00	175.00
Tarred paper	2,000 sq. ft.	@	0.05	100.00
Asphalt	2 bbls.	@	10.00	20.00
Bronze name plate 1 ft. 3 ins. × 2 ft. 0 in.	1	@	50.00	50.00
6-in. Conduit	1,400 ft.	@	0.50	700.00
6-in. Water main	460 ft.	@	1.00	460.00
⅝-in. Clamps	500	@	0.25	125.00
Curbing	690 ft.	@	0.25	175.00
Coffer-dams (estimated)	3	@	1200.00	3,600.00
Arch centers (estimated)	3	@	1000.00	3,000.00

Removing old span				500.00
Temporary bridge	350 ft.	@	2.00	700.00
				$46,478.50
General Expense, 6 per cent				2,788.70
				$49,267.20
Profit, 12½ per cent				6,158.30
				$55,425.50

The bridge designed for crossing Sullivan Gulch in Portland, Ore., by H. W. Holmes, M. Am. Soc. C.E., is a very successful attempt to have a bridge at East Sixteenth Street that will be a piece of architecture and at the same time serve the same utilitarian purpose as the one at East Twenty-first Street. The main arch has a clear span of 160 feet and is of twin rib reinforced concrete design. It carries a 40-foot roadway and two 10-foot sidewalks. The bridge (Fig. 466) is practically symmetrical, is well balanced, beautifully detailed and is in all respects one of the best designs for such a structure that has ever been prepared.

EAST SIXTEENTH STREET CONCRETE ARCH, PORTLAND, ORE.

Concrete "*A*" (1–2–4), 3840 yds.				
Cement.....1.57 bbl. @ 2.00 = $3.15				
Sand.......0.50 yd. @ 1.40 = .70				
Gravel.....0.86 yd. @ 1.40 = 1.22				
Water = .08				
Royalty spouting = .10				
Plant = .60				
General Exp = .60				
Total $6.45				
Arch rings conc.+labor+forms+ctrs. 6.45+0.84+1.50+4.00	1,000 yds.	@	$12.79	$12,790.00
Columns and girders conc.+labor+forms+shoring 6.45+0.84+2.00+2.00	2,300 yds.	@	11.29	25,967.00
Roadway slab conc.+labor+forms+shoring+w.p. 6.45+0.84+2.00+1.50+0.30	433 yds.	@	11.09	4,801.97
Sidewalk slab conc.+labor+forms+shoring 6.45+0.84+2.00+1.50	107 yds.	@	10.79	1,154.53
6-in. Tile drain	350 ft.	@	0.05	17.50
Sheet lead	3,200 lbs.	@	0.09	288.00
Bronze plate	6,550 lbs.	@	0.35	2,292.50
Conduit	2,064 ft.	@	0.25	516.00
Cost, 47,827.50÷3840 = 12.45 per yard				$47,827.50

Fig. 466.—East Sixteenth Street Bridge, Portland, Ore.

Concrete "*B*" (1-3-5) 2050 yds.

Cement.....1.20 bbl. @ 2.10 = 2.52				
Sand.......0.50 yd. @ 1.40 = .70				
Gravel.....0.85 yd. @ 1.40 = 1.19				
Water.................... = .08				
Royalty spouting.......... = .10				
Plant.................... = .60				
Gen. Ex.................. = .60				
Total.................. = 5.79				
Arch footings..............	1,359 yds.	@	6.54	8,887.86
conc.+labor+forms 5.79+0.50+0.25				
Columns, footings............	206 yds.	@	6.54	1,347.24
conc.+labor+forms 5.79+0.50+0.25				
Abutments..................	485 yds.	@	8.38	4,064.30
conc.+labor+forms 5.79+0.84+1.75				
Total..................				$14,299.40

Cost, 14,299.40÷2050=6.97 per yard

Reinforcing steel:

Struct. steel..................	137,000 lbs.	@	0.05	6,850.00
Placing..................	137,000 lbs.	@	0.005	685.00
Bars..................	403,000 lbs.	@	0.03	12,090.00
Total..................				$19,625.00

19,625÷540,000=.036 per pound

Concrete piles................	1,000 ft.	@	1.50	1,500.00	
Concrete rail................	688 ft.	@	1.75	1,204.00	
"*A*" excavation..............	300 yds.	@	2.00	600.00	
"*B*" excavation..............	4,100 yds.	@	0.80	3,280.00	
Pavement..................	1,500 sq. yds.	@	2.00	3,060.00	
Light posts..................	16	@	50.00	800.00	
				$10,444.00	
Total cost..................					$92,195.90
General expense additional..................					1,400.00
					$93,595.90
Profit, 12½ per cent..................					11,699.50
					$105,295.40

The bids received for building scows will vary more than on almost any other class of timber construction, many bidders counting on throwing the material together in almost any way to make the form of a scow, thus causing the owners endless expense in repairing and recalking.

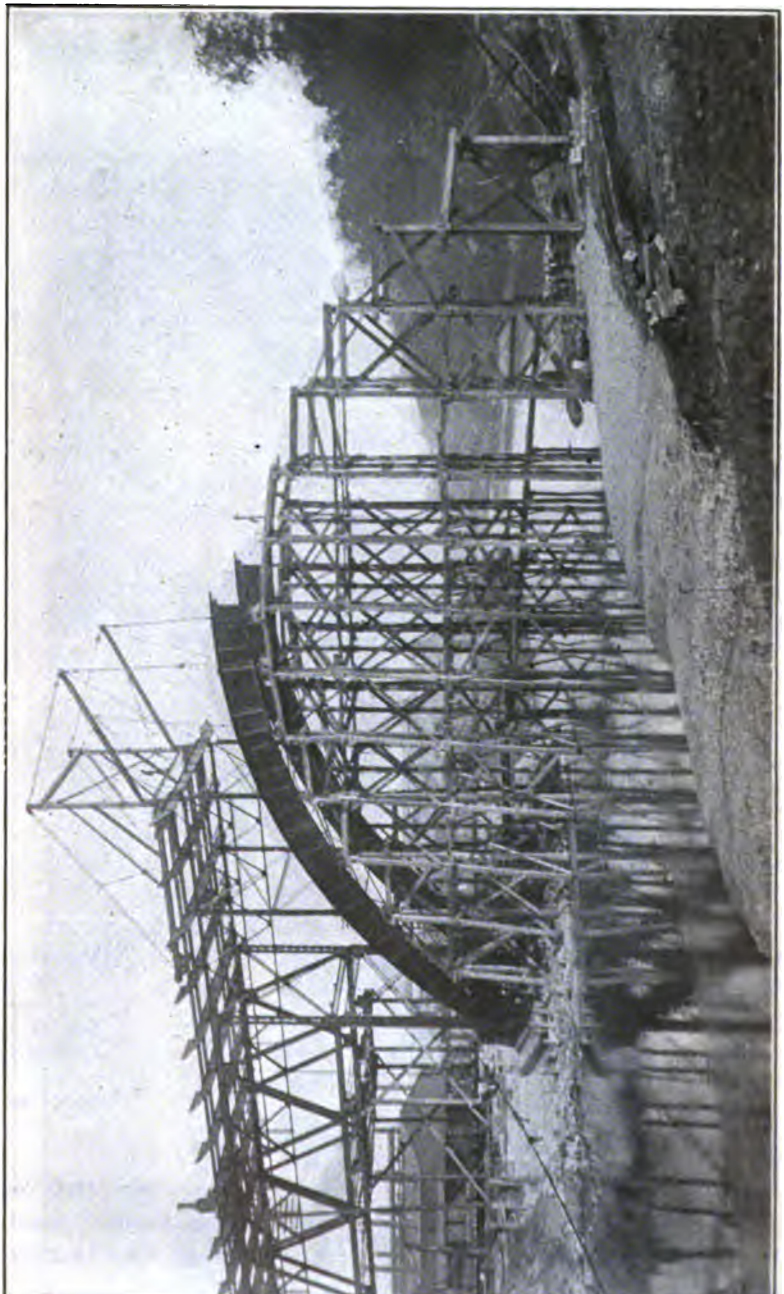

FIG. 467.—PLATE GIRDER ARCH ERECTION SHOWING SKEW-BACK.

The estimate given is perhaps higher than most persons would figure, but to obtain a perfectly good piece of ship carpentry, enough must be spent to obtain first class workmanship. By using cheaper lumber and rougher workmanship, the net cost might be reduced about $1500.00, and by substituting longitudinal bulkheads in place of the solid ones, a still further reduction can be made.

SCOW OR LIGHTER 100×32×10 FT. DEEP

Lumber delivered	112 M.	@	$25.50	$2,856.00
Framing and placing	112 M.	@	20.00	2,240.00
Metal:				
Drifts 1×30 ins	9800 lbs.	@	2.70	264.60
Drifts $\frac{7}{8}$×30 ins	6800 lbs.	@	2.70	183.60
Spikes galvanized	3000 lbs.	@	5.90	177.00
Spikes common	200 lbs.	@	5.00	10.00
Machine bolts	540	@	0.20	108.00
Clinch rings	300	@		5.00
Nails				9.00
Washers				5.00
Rods 66, 1 in. by 10 ft.	1800 lbs.	@	5.60	100.80
Corner irons and straps	1450 lbs.	@	7.00	101.50
Knees				40.00
Paint { 2 cases copper / 20 gals. mineral }				62.00
Felt (and tacks)	20 rolls	@	2.65	53.00
Oakum	28 bales	@	3.75	105.00
Pitch	4 bbls.	@	5.00	20.00
Cement	1 bbl.	@	3.25	3.25
Labor calking and pitching				550.00
				$6,893.75
General Expense, 6 per cent				413.60
				$7,307.35
Profit, 12½ per cent				913.40
				$8,220.75

The estimate of the Navy Quay Wall is for one in shallow water, and built mostly during low tide, although a daily variation of 18 feet in the tides, made the work much more expensive than would appear from an examination of the cross-section in Fig. 411. The concrete was figured at a time when cement was abnormally high, and the use of the galvanized cast iron nosing added very greatly to the expense of the wall. The cost per lineal foot, however, is very reasonable for such construction. The cost of labor on the concrete

covered the plant cost and the cost of pile driving covered the cost of driver.

QUAY WALL AT BREMERTON, WASH., REINFORCED CONCRETE

685 Feet Long

Excavation:			
Above low tide	400 yds. @	$1.00	$400.00
Below low tide	1,500 yds. @	2.00	3,000.00
Sheet piling:			
Wakefield, 6 ins. × 16 ft. (16.00+14.00)	80 M. @	30.00	2,400.00
Wales (16.00+6.00)	13 M. @	22.00	286.00
Guide piles	1,800 ft. @	0.08	144.00
Driving piles	110 @	5.00	550.00
Bolts	23,000 lbs. @	0.06	1,380.00
Removing sheet piles, etc			985 00
Foundation piles:			
Piling	6,500 ft. @	0.08	520.00
Driving piles	430 @	3.00	1,290.00
Concrete 1–3–6	800 yds. @	8.70	6,960.00
cem., 3.00+s., 0.50+g., 1.00+forms, 2.20+labor, 2.00			
Concrete 1–2–4	900 yds. @	11.00	9,900.00
4.45+0.45+0.90+2.20+3.00			
Concrete 1–2 mortar face	120 yds. @	14.40	3,168.00
Reinforcing steel (3.10+0.80)	120,000 lbs. @	3.90	4,680.00
Nosings:			
Galvanized cast iron	25,000 lbs. @	8.50	2,125.00
Bolts	1,200 lbs. @	36.00	432.00
Lead	800 lbs. @	16.00	128.00
			$38,348.00
General Expense, 6 per cent			2,300.90
			$40,648.90
Profit, 12½ per cent			5,081.10
Cost per lineal foot, $66.80			$45,730.00

The author has for many years used a form of daily report for construction work, which tells at a glance just what has been accomplished on each piece of work; and with these arriving by mail every morning from various scattered crews, one is enabled to know just where it is necessary to apply remedies for too high costs.

The form of daily report from a quarry job is given below and tells plainly just what each kind of work in each section of the quarry

was costing and shows also the movement of scows and tugs, as well as other needful information.

N. P. RY. WATERMAN QUARRY DAILY REPORT, OCTOBER 28, 1913

Daily Report.	No. Men.	Total Payroll.	Amt. Done.	Unit Cost.
862 Plant				
863 Plant repairs	5	$17.50		
864 Drilling and blasting, No. 1	4	13.20	335	0.04
No. 2	8	28.42	108	0.05
865 Loading, No. 1	52	130.75	335	0.39
No. 2	38	108.70	608	0.18
Mucking, No. 1	15	47.95	335	0.14
No. 2	10	26.75	608	0.04
866 Transportation	2	6.67		
Scow repairs	3	14.44		
869 Camp				
Supt. and time keeper	2	13.33		
Launch "Ohio"	2	8.24		
Total men working	128	$415.95	943	$0.44
Material loaded				
Scow March Total N. P.			191	
" Wash. No. 3 "			144	
" Chesley No. 8 "			283	
" W. T. & B. No. 1 "			325	
Coal				0.03
Powder				0.03
Time tug departed, *Defender*				
No. Scows, 3, 4.30 A.M. Oct. 29, 1913.				
Empty scows on hand, 5				
Weather, fair.				

The method of figuring the items going to make up the cost of a pile, brush and stone river dike is shown in the following estimate. The cost of all the items will vary greatly from the figures given, with a change of locality and conditions, so that only the forms and general cost is of value in estimating on other work. For each case it will be necessary to determine the number of scows required, the number a tug can tow, the delays from repairs and weather, and many other things affecting the costs. The rip-rap rock in the case given had to be laid up on the face to a 1 to 1 rough slope; if it was only to be dumped to this slope the cost would be less, and if laid up to a smooth face like paving, the cost would be much greater.

GOVERNMENT RIVER DIKES

Stone, broken, sorted and loaded	$0.70
Scows (6 per day @ $6.00)	0.18
Towing, 30 miles	0.15
Tow by launches in river	0.05
Unloading and placing	0.25
	1.33
General Expense, 6 per cent	0.08
	1.41
Profit, 10 per cent	.14
Total per cubic yard	$1.55
Brush fascines on scows	$2.10
Scows (4 per day @ $6.00)	0.50
Towing	0.35
Unloading and placing	0.95
	$3.90
General Expense, 6 per cent	0.23
	$4.13
Profit, 10 per cent	0.42
Total per cord (fir)	$4.45
Willow brush, fascines on scows	$2.75
Scows	0.50
Towing	0.50
Unloading and placing	1.00
	$4.75
General Expense, 6 per cent	0.29
	5.04
Profit, 10 per cent	0.50
Total per cord (willow)	$5.54
Piles for dike, delivered	$0.060
Towing	0.005
Booming	0.007
Moving	0.003
Driving	0.070
Cut-off	0.015
	$0.160
General Expense, 6 per cent	0.010
	$0.170
Profit, 10 per cent	0.017
Total per foot driven and cut-off	$0 187

Lumber rough plank at mill	$10.00
Towing	1.00
Framing and placing	2.50
	$13.50
General Expense, 6 per cent	0.81
	$14.31
Profit, 10 per cent	1.44
Total per thousand in place	$15.75
Spikes f.o.b. per 100 pounds	$3.50
Freight	0.10
Placing	0.10
Waste	0.17
	$3.87
General Expense, 6 per cent	0.23
	$4.10
Profit, 10 per cent	.41
Total per 100 pounds in place	$4.51
Wire for fascines	$2.55
Freight	0.10
Placing	0.20
Waste	0.13
	$2.98
General Expense, 6 per cent	0.18
	3.16
Profit, 10 per cent	0.32
Total per 100 pounds in place	$3.48

Summary:

Stone	25,000 yds.	@	$1.55	$38,750.00
Brush (fir)	11,000 cds.	@	4.45	48,950.00
Brush (willow)	2,000 cds.	@	5.54	11,080.00
Piles	125,000 ft.	@	0.187	23,375.00
Lumber	175 M.	@	15.75	2,756.25
Spikes	3,000 lbs.	@	4.51	135.30
Wire	7,200 lbs.	@	3.48	250.56
Total, 3.5 miles				$125,297.11

The daily reports following, show the work done on two separate days in building a pier and doing some clam-shell dredging at a navy yard, and are only given to represent the method of reporting the daily results of the work, as they do not show various other labor items entering into the cost of the work. None of the costs for material is shown, nothing for fuel, repairs and other items, including general expense.

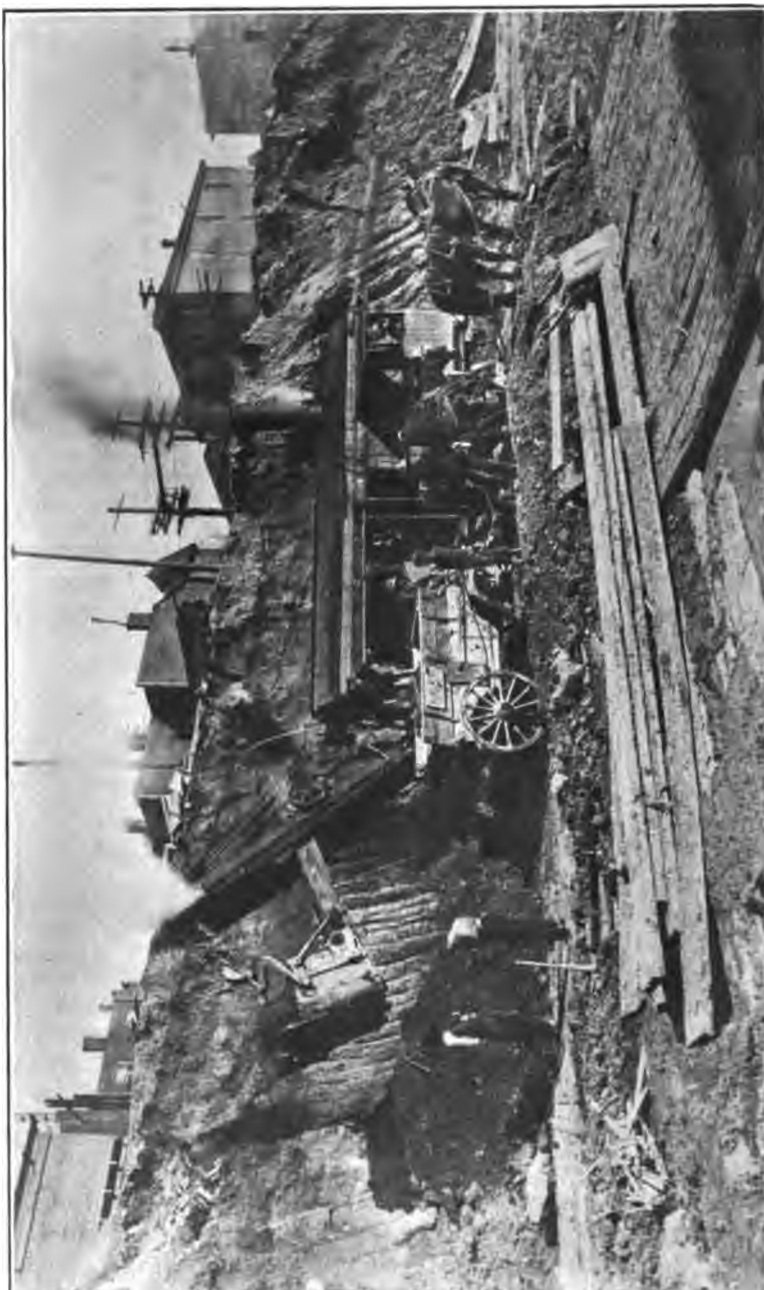

Fig. 468.—Steam Shovel Loading into Wagons.

DAILY DREDGE REPORT, NAVY YARD

TWENTY-FOUR HOURS ENDING 7 A.M. MARCH 25, 1913

No.	Employment.	Hours.	Rate.	Amount.
1	Captain			$5.65
2	Levermen			9.97
2	Engineers			7.10
2	Firemen			5.80
..	Oilers			
3	Deckhands			7.74
..	Scowmen			
1	Blacksmith			3.50
1	Helper			2.75
..	Carpenter			
..	Towboatmen			
..	Cook			
..	Coal passers			
..	Watchmen			
12	Total men			42.51

Character of material.............................. Sand and gravel
Average depth.............................. 10 ft.
Number feet moved.............................. 22
Average width cut.............................. 40 ft.
Average depth cut.............................. 10 ft.
Number hours run.............................. 13
Yards excavated.............................. 330 (day 170, night 160)
Number scows moved.............................. 3
Time and cause, each delay

Day	20 mins.	Oiling
	40 mins.	Dumping
	20 mins.	Lunch
	40 mins.	Changing cables on swinging engine
Night	20 mins.	Oiling
	20 mins.	Lunch
	20 mins.	Changing scows.
	mins.	
	mins.	
	mins.	

Total delay 3 hours.
Tons coal used, 2.
Weather clear.

....
Captain.

PUGET SOUND NAVY YARD DAILY REPORT, MARCH 24. 1913

Daily Report.	No. Men.	Total Payroll.	Amt. Done.	Unit Cost.
Monday				
Removing old deck				
Old piles				
839 Dredging, tug, $20.00				
Labor, 42.51	12	$62.51	330 yds.	$ .19
Pier deck towing				
841 Caps and stringers	8	20.31		
Unloading				
Laying				
Pier piles towing				
Boom	1	4.00		
Driving	6	13.62	13 piles	$1.05
Cut-off	5	16.75		
Track unloading				
Laying				
Total men working	28			
Work complete				
Material received				
Working on				
Making cut-offs and fitting *spur* piles				
Weather, clear.				

PUGET SOUND NAVY YARD, APRIL 14. 1913

Daily Report.	No. Men.	Total Payroll.	Amt. Done.	Unit Cost.
Monday				
Removing old deck				
Old piles				
839 Dredging. Tug $20.00	14	$70.24	680 cu. yds.	$0.10 3/10
Labor 50.24				
Pier deck towing				
Unloading				
Laying	5	19.50		
Pier piles towing				
Boom	2	7.50		
Driving fender piles	7	27.44	37 piles	$.77
Cut-off	4	14.00		
Track unloading				
Laying				
Total men working	32			
Work complete				
Material Received:				
Scow of coal, 37 tons.				
Weather fine.				

DAILY DEDGE REPORT, NAVY YARD

TWENTY-FOUR HOURS ENDING 7 A.M., APRIL 15, 1913

No.	Employment.	Hours.	Rate.	Amount,
1	Captain	....	....	$5.83
2	Levermen	....	....	10.33
2	Engineers	....	....	7.33
2	Firemen	....	....	6.00
..	Oilers	....	....	
4	Deckhands	....	....	10.50
..	Scowmen	....	....	3.50
1	Blacksmith	....	....	2.75
1	Helper	....	....	4.00
1	Carpenter	....	....	
..	Towboatmen	....	....	
..	Cook	....	....	
..	Coal passers	....	....	
..	Watchmen	....	....	
14	Total men	....	....	50.24

Character of material .... Gravel
Average depth .... 12 ft.
Number feet moved 38
Average width cut .... 40 ft.
Average depth cut .... 12 ft.
Number hours run 16
Yards excavated .... 680 (day 410, night 270)
Number scows moved 5
Time and cause, each delay

Day	1.30 mins.	Oiling, putting new guide wheel in bucket
	.30 mins.	Changing scows
	1. mins.	Lunch
Night	1. mins.	Oiling
	.30 mins.	Changing scows
	1. mins.	Lunch
	.30 mins.	Changing scows
	mins.	
	mins.	2 hours overhauling machinery
	mins.	

Total delay, 6 hours.
Tons coal used, 2
Weather, fine.

....
Captain.

The cost sheet of a steam shovel job, where the loading was done into wagons is given below and shows all the items chargeable to such a piece of work. The general expense item is higher than usual, as it includes a rental charge for the shovel, as set forth in the agreement and which is in reality part of the profit.

The costs for dipper-dredge work should be kept in like detail in the ledger, although for floating plant the repairs, rebuilding and depreciation are much larger and more important items. The

FIG. 469.—DIPPER WITH TEETH FOR DREDGING HARD-PAN.

unit cost is determined when the average daily cost of operating the plant is known, by dividing this cost by the number of yards the plant will excavate per day, as is shown by records of similar work as modified by the judgment of the engineer and estimator, owing to the size of the work, character of the material, and the local conditions.

To give this information properly would require a separate chapter and this is not within the scope of the present book, but the author hopes at some future time to cover all this for all kinds of dredges in a separate volume.

COST OF STEAM SHOVEL WORK. HARD CLAY AND GRAVEL

Date.	No. Days.	Total Excavation.	Labor.	Fuel.	Repairs.	Supplies.	Oil and Waste.	Water.	General Expenses.	Total Cost per Month.	Cost per Yard per Month.
June	21	19,562	655.60	129.45	54.95	32.96	7.58	6.00	192.00	1078.54	0.0551
July	25	21,600	665.15	166.05	148.79	51.10	8.78	6.00	200.00	1245.87	0.0577
August	26	20,210	692.47	153.35	152.49	45.88	9.53	6.00	216.00	1275.72	0.0631
September	1	875	22.48	8.45	0.00	0.00	0.00	0.00	8.00	38.93	0.0445
Totals	73	62,247	2035.70	457.30	356.23	129.94	25.89	18.00	616.00	3639.06	0.0583
Average per day		852									
Cost per day			27.88	6.27	4.88	1.77	0.36	0.25	8.44		
Cost per yard			0.032	0.0073	0.006	0.002	0.0035	0.0002	0.01	or approx.	0.06

The items going to make up the cost of a suction dredging job and which should be kept separate on the ledger account of each piece of work are labor, fuel, pipe, new machinery, material, repairs, renewals, supplies and incidentals. To these must be added depreciation, general expense, and in some cases the cost of levees, bulkheads and other shore work.

The two items of renewals and depreciation are in a sense synonomous or at least interdependent to the degree that renewals may

FIG. 470.—COYOTE SHOT, 40,000 YARDS OF ROCK IN THE AIR.

be made to an extent that depreciation after the first year becomes a very small quantity, but the two taken together form a very large, but often neglected item of cost.

When the engineer or estimator has not had the necessary experience to properly judge and estimate the average daily cost and output for any particular piece of work, he should consult some one who has or else obtain preliminary figures and eventually bids from firms having a wide experience in dredging.

APPENDICES

SELECTIONS FROM SPECIFICATIONS

APPENDIX I

SPECIFICATIONS FOR COFFER-DAMS AND FOUNDATIONS, OHIO RIVER MOVABLE DAMS

MAJOR W. H. HEUER, U. S. Engineer

GENERAL DESCRIPTION

The site of Dam No. 2 is on the Ohio River, distant from Pittsburgh, Pa., 10½ miles, and adjacent on the right bank to the Pittsburgh, Ft. Wayne and Chicago Railway. It has Neville Island on the left bank, and is accessible by street cars from Pittsburgh.

The lock is to be located on the left bank of the Ohio River, immediately behind Merriman's dyke. It will be in general dimensions the same as locks Nos. 1 and 6, viz., 110 feet wide and 600 feet long.

SPECIAL DESCRIPTIONS

The river bed at No. 2 consists of gravel throughout, and the excavations will be made to a depth sufficient to insure a permanent and enduring foundation, which will ordinarily be 14 feet below the gate sill, but may be otherwise, as the engineer, in his judgment, may direct.

The work will conform to the drawings exhibited, and to such others, in explanation of details or modifications of plans, as may be furnished from time to time during construction.

CONTRACTOR TO FURNISH ALL MATERIAL AND WORK.—It is understood and agreed that the contractor, under his contract prices for work in place, is to furnish and pay for all materials, stone, cement, sand, earth, timber, material for coffer-dam and protection cribs, excavation, lock-filling and discharging valves (set in masonry), flushing valves, anchor bolts, lock-gate tracks, and everything entering into or connected with either the permanent or temporary construction, and he is also to supply and pay for all work, skilled and other-

wise, required to prepare and place the materials, and complete the work according to the drawings and these specifications.

CONTRACT TO INCLUDE.—The contract will cover the construction and completion of the foundation, masonry and timber work of the lock, including both land and river walls, the gate-recess walls, the foundations of the lock-gate tracks, the guiding walls above and below the lock, the pipe and flushing conduits, the drift chute, the foundations for the power-house and lock-keepers' residence, and every such other permanent construction as shall be shown upon the drawings. It shall also include the clearing of the land necessary for the proper execution of the work embraced in this contract, all pumping and bailing, dredging and excavation, puddling and embankment, the construction of all coffer-dams, stone masonry, concrete and brick masonry, timber work and iron work, and all such other work which, in the judgment of the Engineer, is necessary and included in the proper completion of the contract.

TOOLS, MACHINERY, BUILDINGS, ETC.—The contractor, without cost to the United States, shall furnish all appliances, dredges, pumps and pumping machinery, boats, tools, derricks, tramways, foot-walks, roads and landings, and all needful temporary buildings and shops.

COFFER-DAMS

SHEETING.—The coffer-dam, about 1500 feet in length, shall be built as shown generally by the drawings exhibited, and as directed by the Engineer. It shall be 14 feet high above the sill of the lock, and shall consist of two walls or rows of plank sheeting, spaced 12 feet apart in the clear, driven or set firmly from 1 to 2 feet into the river-bed, and supported laterally by horizontal longitudinal stringers, the latter being spaced at varying intervals, increasing in width from the bottom to the top, and to be sufficiently and firmly bolted together transversely with iron rods passing through the coffer-dam horizontally from the rows of stringers on the one side to the corresponding rows on the other, against which the vertical plank sheeting shall be securely spiked.

FILLING AND DECKING.—The interior, or space between the walls of sheeting, shall be filled with heavy dredged river-bed or other material not liable to wash, and to be covered over with a suitable decking of plank (to protect it from injury in case of being submerged by floods), all complete as shown on the drawings.

PILING AND CRIBS TO PROTECT.—At the upper outer corner of the coffer-dam shall be placed a crib built of framing timber and filled with riprap stone; from the upper corner of the crib, at an angle of 45° with the axis of the current, a line of piling, spaced 5 feet apart, firmly bolted together with waling-pieces, shall be driven to the shore to form a protection to the coffer-dam; also outside and along the coffer-dam, from the upper outer corner to the lower corner, clusters of piles, firmly bound or bolted together, shall be driven at intervals of about 80 feet. The tops of all piling shall be sawed off to a uniform height of 2 feet above the coffer-dam. Protection cribs shall be placed at such other points along the coffer-dam as may be shown upon drawings.

HOW PAID FOR.—Bidders will state a price per lineal foot of coffer-dam completed. No payment will be made for any portion thereof until the entire coffer-dam is conpleted. Drawings will be furnished, showing the general type of the coffer-dam and its manner of construction, and every detail necessary for intelli-

gent bidding. Should any work on the outside of the coffer be necessary, such as gravel filling or riprapping, it shall be paid for at the price bid for gravel filling, riprapping, etc. If, owing to the nature of the river bed, it shall be found impossible to drive the plank sheeting to the required depth, then the contractor, after driving the sheeting as deep as possible without injury, and in lieu of driving it to its full depth, shall fill around the outside of the walls with the same material as is used in filling the coffer-dam, to the height of 4 feet above the surface of the river-bed, and for which no extra compensation will be allowed.

REMOVAL OF.—The contractor will be required to remove the coffer-dam and its belongings at his own cost. The time and manner of the removal of the coffer-dam, or any part thereof, and the place to deposit the materials shall be prescribed by the Engineer.

TO BELONG TO THE UNITED STATES.—It is understood and agreed that the payments made for the coffer-dam, including the crib and pile protection, shall cover the entire cost thereof to the United States, and by virtue thereof they shall become the property of the United States. The contractor, however, must maintain the same and make all needed repairs to same during the existence of the contract, without expense to the United States.

DEPOSIT WITHIN THE COFFER-DAM.—Material washed or left in the space inclosed by the coffer-dam by freshets shall be removed by the contractor, as directed, at his price for common excavation, which price shall cover all necessary cleaning and scrubbing. No payment will be made, however, for removing material washed into the inclosure from the coffer-dam itself or from any deposit made by the contractor on or above the works.

MATERIAL AND WORKMANSHIP

TEMPORARY PILING shall include all piles driven for the protection of the coffer-dam and " deadmen " for derricks. They shall be of good quality, round oak timber, not less than 12 inches diameter at the butt, and of length varying from 20 to 25 feet, and longer if necessary.

SHEET PILING.—In excavating for foundations, should quicksand or fine sand carrying water be encountered, close sheet-piling will be required to be driven to whatever extent the Engineer may direct.

SHEETING.—The sheeting shall include the walls and decking of the coffer-dam, including the stringers; also such shoring as may be directed by the Engineer to remain in the finished structure. It shall consist of the best quality of hemlock obtainable, and must be in all cases satisfactory to the Engineer in charge.

GRAVEL OR EARTH FILLING.—Gravel or earth filling will include all material used in filling the land-wall inclosure, back of the guiding walls, etc. It does not include any filling in the construction of the coffer-dam.

STONE FILLING shall include all stone placed in the protection cribs or any riprap stone ordered for the protection of the work.

CRIBWORK shall be built of hemlock framing timber framed together in square bins and securely bolted together by iron drift-bolts. The interior of the cribs shall be filled with riprap stone, and should the Engineer deem it necessary such riprap stone shall be placed on the outside of the crib. The whole to be built as shown by the drawings.

FRAMING TIMBER.—For all temporary cribwork, also the permanent crib at the head of the upper guiding wall, framing timber shall be used. No stick shall be less than 10″×10″ in section.

"Framing timber" is a commercial term for a class of timber hewn to various sizes.

EXCAVATION

TO INCLUDE.—It shall include the removal of all gravel or other material to the depth required for the lock and its upper and lower entrances, the gate recesses, Poiree-dam and gate-track foundations, for the foundations of all walls, and for all conduits or wells, and all such other material as may be found necessary in the judgment of the Engineer to be removed for foundations and otherwise in permanent construction. It will include all dredging and all material excavated of whatever nature, however removed, for foundations and for site of coffer-dam.

LINES, SLOPES, AND GRADES FOR.—All excavations shall conform to such lines, slopes, and grades as may be given by the Engineer, and anything taken out beyond such given limits will not be paid for by the United States.

MATERIAL TO BE DEPOSITED.—Excavated material is to be deposited as and where directed by the Engineer. It shall be deposited in such manner as not to interfere with present or proposed navigation. Material of any kind deposited by the contractor in absence of, or in disregard of, instructions, shall, if required by the Engineer, be removed by the contractor at his own cost.

SHORING.—All excavation for foundation shall be securely shored and thus maintained until the foundation has been sufficiently advanced to dispense with the same, when it may remain or be removed at the discretion of the Engineer.

DREDGES AND PUMPS.—The contractor will be required to employ, at the same time, not less than two suitable steam dredges at excavating and filling; and for pumping he must keep at least three good sufficient pumping outfits, with pumps, engines, and boats complete, in or always ready for operation. The dredges must be equipped to do effective work to a depth of 28 feet.

FOUNDATIONS

CHANGES OR MODIFICATIONS OF.—The character of the river-bed and of the proposed foundations for the different parts of the work is shown in general on the drawings and cross-sections exhibited, and it is understood that the United States shall have the power to make any changes in the plans of the foundations that may, in the judgment of the Engineer, be considered advisable after examinations made, as the excavations proceed within the coffer-dam after it is pumped out, and it is understood and agreed that the contractor shall have or make no claim against the United States on account of any such changes in or modifications of the plans of the foundations, or on account of any increase or decrease in the depth of same, under or over those referred to herein or shown on the drawings exhibited.

MASONRY

CEMENT.—Cement will be of uniform quality, setting well both in air and water, and free from anything that will cause the mortar to swell, crack, or scale. It shall be put up in strong, sound barrels, well lined with paper so as to be reasonably protected from air and moisture. The average net weight of the barrels shall be not less than 265 pounds, unless expressly so stated in the proposal. Each barrel must be labeled with the name of the brand and of the manufacturer.

In general, ten barrels of every one hundred will be tested.

The cement must stand the following tests: Fineness—At least 85 per cent must pass a sieve of 6400 meshes to the inch. Setting—Cement must be modeerately slow setting; it must not begin to set within fifteen minutes, as determined by Vicat needle $\frac{1}{12}$ inch in diameter with $\frac{1}{4}$ pound load, and it shall not bear weight of one pound on wire $\frac{1}{24}$ inch in diameter within thirty minutes, but must bear such weight within one hour and a half. Strength—The minimum tensile strength per square inch of briquettes of neat cement mixed with about 33 per cent of water by weight, and exposed in air for one hour, and the remainder of 24 hours in water, shall be not less than 50 pounds; with longer time, whether in air or water, there must be a decided increase of strength; it must also test to the satisfaction of the Engineer when mixed with sand. The tests for setting will be made at a temperature of air and water of about 75° Fahrenheit. All other tests will be made at a temperature above 60° Fahrenheit. The cement will be subject to inspection at all times, and must be kept well housed.

SAND.—The sand used must be clean, sharp, washed, river sand, satisfactory to the Engineer.

MORTAR.—To be composed generally of two parts of sand to one of cement; when required, and whenever thought necessary by the Engineer, it shall be made richer. It must be thoroughly mixed and used before it has begun to set. If required by the Engineer, the mortar beds will be protected from the sun.

POINTING.—All face work is to be pointed, as fast as the work progresses, with stiff mortar, mixed one of sand to one of Portland cement, thoroughly hammered in and finished with proper tools; before the final acceptance of the work all face masonry which at that time does not appear properly pointed shall be repointed by the contractor to the satisfaction of the Engineer, without extra cost.

FROST.—Masonry will not be executed during freezing weather, nor when, in the judgment of the Engineer or his agent, it is likely to freeze before the mortar shall set. To guard against injury from frost, all new and unfinished work shall be properly protected by the contractor at his own cost.

VOIDS AND OPENINGS.—Due regard shall be had in the construction of all masonry walls to leave all necessary voids or openings for conduits or wells, or for such other purposes as may be required by the Engineer.

ASHLAR.—It shall comprise such part of the walls as is built of stone, with point-dressed face, and beds, and joints smoothly and squarely dressed.

QUALITY OF STONE.—All stone shall be perfectly sound, strong, hard, free from injurious seams, and in all respects satisfactory to the Engineer. Stone to be such as can be truly wrought to such lines and surfaces, whether curved or plain, as may be required. No stone shall be used which weighs less than 135 pounds to the cubic foot.

Samples of Stones. Each bidder must deposit at this office, all charges prepaid, before the bids are opened, a 6-inch cubical block of the stone he proposes to furnish, and state the quarry from which it was obtained. The quality of the stone must be at least equal to that of the sample. The sample must be truly squared, and dressed on four sides; one side must be hammer-dressed, one side smooth-dressed and rubbed, and one side pitch-dressed. The other side is to be left with quarry face.

Stone May be Rejected.—The United Stated reserves the right to reject any stone not deemed suitable, or which, during the execution of the contract, shall be found defective. The beds of the stone must be their natural quarry beds. No lewis or dog holes, letters, or marks of any kind will be allowed on any dressed face of stone, but each face shall have left on it a boss for lifting, to be removed by the contractor after the stone has been set.

Dressing of Stone.—Stone must be accurately cut, square, and true, and the faces must be pitch-draughted and point-dressed to a plane with the draught, forming an approximately smooth surface. The beds must be smoothly and squarely dressed, full length and width. The vertical joints must be dressed to a depth of not less than 18 inches from the face, and the allowance for joints must not exceed ⅜ inch. One-third of the stone in each course must be headers. All stones not accurately dressed will be rejected. All dressed stone must have the dimensions plainly marked on one end.

Dimensions.—The cut-stone stretches must be not less than 3 feet nor more than 5 feet long, and their width must not be less than 1⅓ times the height of the course to which they belong. The width of the headers must be not less than 1⅓ times their height, and their length must be at least double their breadth, unless otherwise ordered. The thickness of course includes the joint, which will be ⅜ inch.

Laying Stone Masonry.—The faces of the wall shall be accurately laid to the lines indicated on the drawings, or as directed by the Engineer. All stones to be well laid to proper lines, in full beds of mortar, and settled in place with a wooden maul; the use of grout is prohibited. No dressing, except in special cases, and by permission of the Engineer, will be allowed on backing after it is laid in the wall. The bond of stone shall in no case be less than 9 inches. The walls will be laid in horizontal courses throughout, each course to be of uniform height through the wall. Heights and arrangements of courses to be determined by the Engineer. When laying masonry the site for the stone shall be thoroughly cleaned with a scrub-broom and moistened; and the stone shall always be cleaned and well moistened before being set. Not more than three unfinished courses of face stone will be permitted upon the wall at the same time without special permission from the Engineer in each case. Proper machinery must be used in handling the stone; face stone shall not be disfigured by use of plug or grabs. Any stone chipped or spalled shall be rejected. Stones having defects concealed by cement or otherwise will be rejected on that account alone.

Coping.—The coping will be of the same class and quality of stone described in ashlar masonry. It will be carefully and truly cut to forms and dimensions given, from the best stone; it will be crandalled on all outer faces; the exposed edges of the coping to be rounded to a radius of 3 inches and chiseled smooth where required. Beds and vertical joints to be pointed true and full throughout and be laid with ⅜-inch joints.

The coping is to be doweled as required by the Engineer with round iron,

the dowels to be furnished and placed by the contractor. The drilling for and placing of the dowels will be covered by the price for "Bolt Holes in Masonry." The dowels will be set in Portland cement.

RUBBLE STONE

Quality and Dimensions of.—Rubble stone must be sound, hard, and durable, free from seams, scale, earthy matter, and other defects. Rubble stone shall in general be not less than $\frac{3}{4}$ of a cubic foot in size. It must be in fair shape for laying in the face of the walls without dressing. No spalls will be allowed.

Laying.—The stone must be laid on their natural bed in full beds of hydraulic cement mortar, with all joints and voids well filled with mortar. Leveling up under stones with small chips or spalls will not be allowed.

The stone shall be carefully selected for the outer face so as to have vertical joints and present a good face of broken rough masonry.

CONCRETE

Composition of.—Concrete shall be composed of satisfactory cement and river gravel; the latter, should it be of an approved quality, shall be taken from the various excavations of the lock and its walls. This gravel generally has a sufficient volume of sand to fill all voids; should there be a deficiency of sand in any portion of the gravel the contractor will be required to supply said deficiency by good, sharp, washed, river sand. The quantity of cement to be used will generally be about 20 per cent greater than the volume of voids in sand and gravel.

Mixing and Placing of.—The concrete is to be well and rapidly mixed by machinery, as may be required by the Engineer, unless otherwise specified. It will be deposited in layers not more than 8 inches thick; wherever and whenever required, the layers will be thinner than 8 inches, and all thoroughly rammed by such process as the Engineer may approve.

River Wall.—In the river wall of the lock the concrete shall be laid in courses of a thickness corresponding to the adjoining courses of ashlar masonry. It shall be filled in flush with the top of each course before the next course of ashlar above shall be laid.

Before putting in the concrete of any course the bed and adjoining course of ashlar shall be thoroughly wetted so that no dry surface may come in contact with the fresh concrete, destroying its power of adhesion by absorbing its moisture.

In order that the work once began may progress without delay all cut stone needed for the ashlar facing shall be on the ground when the concrete foundation has been completed.

TIMBER IN PERMANENT CONSTRUCTION

To Consist of all timber used in the timber facing of the lock walls and the guide walls; all timber cribbing in the gate-track and Poiree-dam founda-

tions; the oak sheeting at the head of the guide walls; and such other timber in permanent construction as shall be shown upon the drawings.

General Quality and Dimensions.—All timber must be first class, and any of inferior quality will be rejected. Sap-wood in any stick will cause its rejection. The timber must be free from black or rotten knots, wane edges, wind-shakes, dose, or other imperfections. Firm, sound knots, if not too numerous, will not be considered defects. Timber must be full size, true and out of wind, and when required must be sawed large enough to dress down to required dimensions. The timber will be inspected on arrival at the work, and if found to be defective will be rejected.

Oak.—Oak timber must be taken from the best quality live white oak sawed timber.

White Pine.—Shall consist of the best quality of clear white pine obtainable.

Hemlock.—Shall be the best quality of hemlock obtainable.

Framing, Assembling, and Painting.—All timber must be accurately framed, fitted, and assembled, according to detailed drawings and directions. As the timber is framed it shall be painted about the ends and elsewhere as may be required to prevent checking. The paints for this will be furnished and applied by the contractor, and covered in his price for " Timber in Permanent Construction."

Timber Facing, Uprights, and Sheeting shall be constructed of oak, and shall consist of uprights spaced at intervals of 6 feet, center to center, anchored to the concrete masonry by tee-head screw-bolts as shown on drawings. To the uprights shall be bolted, with wrought-iron screw-bolts, oak sheeting 6 inches thick.

Nosing Timber shall extend along the top of the guide wall, forming a cap to the uprights and securely bolted to them, as shown on the drawings. The top surface of the nosing shall be flush with the top of the concrete masonry wall.

Oak Sheeting.—This refers to the sheeting on the upper faces of the protection crib for the upper guiding wall at the upper end thereof. It shall be spiked on and firmly held in place with iron bands or straps bolted to the framing timbers of the crib, if, in the judgment of the Engineer, this may be deemed necessary.

SUPERVISION AND MEASUREMENT OF WORK

Inspection, Rejected Material, etc.—The works will be conducted under the direction of the local or resident Engineer, who shall have power to prescribe the order and manner of executing the same in all its parts; of inspecting and rejecting materials, work, and workmanship which, in his judgment, do not conform to the drawings that may be furnished from time to time, or to these specifications. And any material, work, or workmanship so rejected by him shall be kept out of or removed from the finished work, and no estimate or payment shall be made until such material, work, or workmanship be so removed.

When so required rejected material shall be piled up in sight near the works and kept there until the Engineer gives permission to have it removed.

The United States will keep inspectors on the work who will receive instructions from the resident Engineer. They will have power to object to any materials, work, or workmanship. Any material, work, or workmanship objected

to by the inspectors shall be kept out of or removed from the finished work, unless in each particular case the objections of the inspector shall be overruled by the local or resident Engineer; and, unless the objection be so overruled, no estimate or payment shall be made until such material, work, or workmanship be so removed.

The local or resident Engineer shall have power to overrule or rescind any or all objections or decisions of the inspector.

The decision of the United States Engineer Officer in charge of the works shall be final and conclusive upon all matters relating to the work and upon all questions arising out of these specifications, and from his decision there shall be no appeal.

FAILURE TO PROSECUTE OR PROTECT WORKS.—If at any time the contractor shall refuse or fail to prosecute the work or provide for carrying on the same as directed by the Engineer, or fail to properly protect any part of the work, permanent or temporary, the Engineer shall have power to employ men, to purchase or otherwise provide materials, tools, machinery, etc., and put the work in proper advancement or condition, and the entire cost of so doing shall be deducted from payments to be made under this contract.

COMPLETE WORK REQUIRED.—The contractor is not to take advantage of any omissions of details in drawings or specifications, or errors in either, but he will be required to do everything which may be necessary to carry out the contract in good faith, which contemplates everything complete, in good working order, of good material, with accurate workmanship, skillfully fitted and properly connected and put together. Any point not clearly understood is to be referred to the Engineer for decision.

CHANGES.—Should any changes in the details of the shape, arrangement, or fitting of the parts be deemed necessary or advisable in the progress of the work, they must be made by the contractor, and a fair allowance will be paid for any changes which, in the judgment of the Engineer in charge, materially increase the cost of the work.

MEASUREMENT.—Measurement of all work and material shall be made in place, unless otherwise specified.

COFFER-DAM.—The price per lineal foot of coffer-dam shall include all material, lumber, iron, and gravel entering into its construction. A profile of the location will be furnished, showing a section of the river-bed over which the coffer-dam is located, so that the contractor may estimate the amount of each kind of material required.

PILING.—Temporary piling shall be measured in lineal feet, and measurement shall be allowed for total length of piling used.

SHEETING.—This will include all lumber used for temporary purposes, in shoring of excavations, or for forms necessary to sustain any concrete masonry until it has become sufficiently hardened. Sheeting required by the Engineer to remain in the finished structure shall be paid for at the contractor's price per thousand feet B.M. All temporary sheeting not remaining in the finished structure shall be included in the contractor's unit price for material in place, and no estimate will be made thereof by the Engineer. Coffer-dam sheeting will be included in the contractor's price per lineal foot of coffer-dam.

FILLING.—Gravel filling will be measured in the fill, and will not include any filling placed in the coffer-dam as coffer-dam filling.

Stone filling shall include all riprap work, either temporary or permanent.

EXCAVATION.—Excavation will be measured in excavation by cross-sections.

MASONRY.—All masonry, ashlar, rubble, brick, concrete, etc., will be measured by the cubic yard in place. Prices for masonry will include all required pointing. No payment will be allowed for voids or openings.

BOLT HOLES.—All holes drilled in rock or concrete or other masonry will be measured by the running foot as drilled.

TIMBER IN PERMANENT CONSTRUCTION.—Timber in permanent construction will include all timber used in any part of the permanent construction; unless otherwise particularly specified, it will be classed under the following heads:

Oak in Permanent Construction.

Pine in Permanent Construction.

Hemlock in Permanent Construction.

APPENDIX II

EXTRACTS FROM TOPEKA (KANSAS) MELAN ARCH BRIDGE SPECIFICATIONS

By permission of EDWIN THACHER, M. Am. Soc. C. E.

PILING IN PERMANENT WORK

Piling and lumber for coffer-dams to be sound white oak, yellow pine, or other woods equally good for the purpose, the quality to be acceptable to the superintendent. The piles shall be straight-grained, trimmed close, and have all bark taken off, and shall be at least 10 inches in diameter at the small end and 14 inches in diameter at the butt when sawed off. The heads shall be cut off squarely at right angles to the axis of the pile, and all piles shall be fitted to and driven with a cast-iron head. The piles shall be driven with a hammer weighing not less than two thousand two hundred and fifty (2250) pounds, and until they do not move more than three-eighths ($\frac{3}{8}$) of an inch under a blow of the hammer falling twenty-five (25) feet. No pile shall be driven less than twenty-six (26) feet below low water, and if necessary to attain this minimum depth jets shall be used in addition to hammer. The number and arrangement of the piles for each foundation are shown on the plans, and must be carefully carried out by the contractor. The piles shall be cut off at an elevation of about six (6) inches below low water. A slight variation will be allowed, but no piles must be cut off at a higher elevation. Inspection of piling and lumber, except at bridge site, shall be at contractor's expense.

COFFER-DAMS

After the bearing piles have been driven, a permanent coffer-dam, of the dimensions marked on the plans, of Wakefield (or other equally satisfactory) sheet-piling, shall be used around each foundation. The earth inside thereof shall be excavated to the depth shown on plans and replaced with concrete as hereinafter specified. During the placing of the concrete the water shall be kept out of the coffer-dams, unless the bottom is so porous that it is impracticable in the opinion of the superintendent to do so—in which case some of the concrete may be placed in position by means of chutes, under the direction of the superintendent, until the bottom is well calked, after which the water shall be pumped out and the remaining concrete placed in position. The contractor will be required to make the sides and ends of the coffer-dams watertight, and no leak through them will be considered sufficient cause to require any concrete to be placed by means of chutes.

CENTERING

The contractor shall build an unyielding falsework, or centering, of the form and dimensions shown on the plans; particular care must be taken to drive the piles supporting it to a solid bearing. The estimated load upon each of these piles is twenty (20) tons. The contactor must, however, satisfy himself as to the load each pile will have to bear, and as to its supporting power. In case of any settlement the contractor shall take down and rebuild the centering and arch. The lagging shall be dressed on both edges to a uniform size, so that when laid it will present a smooth surface, and this surface shall be built at the proper elevation to allow for settlement of arch, so that when the centering is struck the arch-ring will come to the elevations shown on plans.

The top surface of the lagging shall be covered with W. Field's Building Paper of medium weight, known as Double Saturated Water-proof Oiled Sheathing Paper (or other equally good) to prevent the concrete from adhering thereto. No center shall be struck until at least twenty-eight (28) days after the completion of the arch. Great care shall be used in lowering the centers so as not to throw undue strains upon the arches, nor shall any center be struck before the adjoining arch has been completed for a sufficiently long time, in the opinion of the superintendent, to be uninjured thereby.

NOTE.—For the above reasons it is probable that the five centers will be in use at the same time.

PORTLAND CEMENT

The Portland cement shall be a true Portland cement, made by calcining a proper mixture of calcareous and clayey earths; and the contractor shall furnish one or more certified statements of the chemical composition of the cement and of the raw materials from which it is manufactured. Only one brand of Portland cement shall be used on the work, except with permission of the superintendent, and it shall in no case contain more than two (2) per cent of magnesia in any form.

The fineness of the cement shall be such that at least 98 per cent shall pass through a standard brass cloth sieve of 74 meshes per linear inch, and at least 95 per cent shall pass through a sieve of 100 meshes per linear inch.

Samples for testing may be taken from each and every barrel delivered, as superintendent may direct. Tensile tests will be made on specimens prepared and maintained, until tested, at a temperature of not less than 60° Fahrenheit. Each specimen shall have an area of one square inch at the breaking section, and after being allowed to harden in moist air for twenty-four hours shall be immersed and retained under water until tested.

The sand used in preparing the test specimens shall be clean, sharp, crushed quartz, retained on a sieve of 30 meshes per linear inch and passed through a seive of 20 meshes per linear inch, and shall be furnished by contractor.

No more than 23 to 27 per cent of water by weight shall be used in preparing the test specimens of neat cement, and in making the test specimens one of cement to three of sand, no more than 11 or 12 per cent of water by weight shall be used.

Specimens prepared from neat cement shall after seven days develop a tensile strength not less than 400 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand (parts by weight) shall after seven days develop a tensile strength of not less than 140 pounds per square inch, and after twenty-eight days not less than 200 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand (parts by weight) and immersed, after twenty-four hours, in water to be maintained at 176° Fahrenheit, shall not swell nor crack, and shall after seven days develop a tensile strength of not less than 140 pounds per square inch.

Cement mixed neat with about 27 per cent. of water, to form a stiff paste, shall, after 30 minutes, be appreciably indented by the end of a wire one-twelfth inch in diameter, loaded to weigh one-quarter pound.

Cement made into thin cakes on glass plates shall not crack, scale, or warp under the following treatment. Three pats shall be made and allowed to harden in moist air at from 60° to 70° Fahrenheit; one of these shall be subjected to water-vapor at 176° Fahrenheit for three hours, after which it shall be immersed in hot water for forty-eight hours; another shall be placed in water at from 60° to 70° Fahrenheit, and the third shall be left in moist air.

Samples of one-to-two mortar and of concrete shall be made and tested from time to time as directed by the superintendent. All cement shall be housed and kept dry till wanted in the work.

Storage rooms and rooms and apparatus for the tests shall be furnished by the contractor, and all tests shall be made entirely at his expense, and under the direction and to the satisfaction of the superintendent.

PORTLAND CEMENT CONCRETE

The concrete shall be composed of clean, hard, broken limestone (or gravel with irregular surfaces) and cement mortar in volumes as hereinafter described. The sand shall be clean, sharp, Kansas River sand, washed *entirely* free from earth and loam. If obtainable, a mixture of coarse and fine sand shall be used. Approved mixing machines shall be used. These machines must be kept clean and no accumulations of old mortar shall be allowed to form in them. The ingredients shall be placed in the machine in a dry state and in the volumes specified and be throughly mixed, after which clean water shall be added and the mixing continued until the wet mixture is thorough and the mass uniform. No more water shall be used than the concrete will bear without quaking in ramming. The mixing must be done as rapidly as possible, and the batch deposited in the work without delay, and before the cement begins to set. Stone must be entirely free from earth and earthy surfaces. Thin splints or leaves of stone, easily broken with fingers, will not be allowed to go into the work. The quality of stone and the crushing must be acceptable to the superintendent.

The grades of concrete to be used are as follows (parts by volume):

For the arches: One part Portland cement, two parts sand, and four parts broken stone (hazelnut size, from one-half inch to one inch), except for the exposed faces and soffits of the arches, which shall have at least one inch in thickness of mortar composed of one part Portland cement and two parts sand.

For the piers, abutments, spandrel and wing-walls: On the exposed surfacer for at least one inch thick, one part Portland cement and two parts sand; for

the next seven (7) inches, one part Portland cement, two parts sand, and four parts broken stone of hazelnut size. For the remaining portions: One part Portland cement, four parts sand, and eight parts broken stone of size to pass through a 3-inch ring, except such portions of the interior of the piers and abutments as are above the top of the cornice, or elevation 15.75 feet above low water, which shall be composed of one part Portland cement, three parts sand, and six parts broken stone which will pass through a $2\frac{1}{2}$-inch ring.

No plastering of surfaces will be allowed nor any practice that will develop planes or surfaces of demarcation other than those hereinafter described. Immediately after the removal of any forms or centers, sand and cement shall be sifted on the surfaces and the surfaces rubbed hard with a float as may be directed by the superintendent.

During warm and dry weather and whenever the superintendent shall direct, all newly built concrete shall be kept well shaded from the sun and well sprinkled with water at the surface for several days or until well set.

There must be no definite plane or surface of demarcation between the facing and the concrete backing. The facing and the backing must be deposited in the same layer and well rammed in place at the same time.

In connecting old concrete with new, in the planes hereafter described, the old concrete shall be cleaned and roughened and soaked with water, and at the points of contact a mortar composed of one part cement and two parts sand shall be used and shall be laid in the same manner as specified for laying the facing.

NATURAL CEMENT CONCRETE

The concrete around piles, to take the place of the earth excavated from the coffer-dams, shall be composed of one part natural cement, equal to the best Fort Scott, Kas., cement, three parts sand, and six parts of broken stone of the size to pass through a 3-inch ring. This concrete may be mixed by hand on platforms adjoining the foundations and shoveled directly into the coffer-dams, care being taken to deposit it in uniform layers of about 6 inches each and to carefully ram each layer.

PIERS, ABUTMENTS, AND SPANDRELS

All piers, abutments, spandrels, and wing-walls shall be built in timber forms. These forms shall be substantial and unyielding, of proper dimensions for the work intended and closely jointed, and all surfaces that come in contact with the concrete shall be smoothly dressed and well oiled with linseed oil to prevent the concrete from adhering to them. That portion next to the exposed faces of the work need not be oiled, but shall be covered with oiled paper, the same as that specified for the centers.

Molds, to form molding and panels, smoothly finished and well oiled with linseed oil, shall be properly placed in the forms so that the finished work will appear as shown on the plans. Extreme care must be used to place them in proper position before placing any concrete or mortar in them.

CONTINUOUS WORK

The following divisions shall constitute sections for continuous work, viz.: Each footing course of piers or abutments; each pier or abutment from footing course to cornice; each pier or abutment from cornice to springing line of arch; each spandrel wall from keystone to pier or abutment; each pier or abutment spandrel wall; that portion of the piers or abutments above springing line of arch shall be considered part of the longitudinal sections of the arch previously described.

Each of the above sections shall be carried on continuously night and day if necessary; that is, each layer shall be well rammed in place before the previously deposited layer shall have time to partially set.

Care shall be taken to make the joints (for expansion) in each spandrel wall over piers as indicated on the plans.

CONCRETE IN COFFER-DAMS

The natural cement concrete in the coffer-dam shall extend from depths marked on plans to 1 foot below low water. Upon this concrete the footing courses of piers and abutments shall be founded.

The sheet-piling of coffer-dams shall be cut off at least down to low-water mark, neatly and evenly, by the contractor before the completion of the work.

APPENDIX III

EXTRACTS FROM KATTE'S MASONRY SPECIFICATIONS

By permission of WALTER KATTE, M. Am. Soc. C.E.

EXCAVATIONS will be classified under the following heads, viz.: Earth, hardpan, loose rock, solid rock, and excavation in water.

EARTH will include clay, sand, gravel, loam, decomposed rock and slate, stones and boulders containing less than one cubic foot, and all other matters of an earthy nature, however compact, excepting only "hardpan," as described below.

HARDPAN will consist of tough, indurated clay or cemented gravel which, in the opinion of the Engineer, requires blasting for its removal.

LOOSE ROCK.—All boulders and detached masses of rock measuring over one (1′) cubic foot in bulk, and less than one (1) cubic yard; also all slate, shale, soft friable sandstone and soapstone, and all other materials excepting rock, solid ledge, and those described above; also stratified rock in layers of not exceeding eight (8″) inches in thickness, when separated by strata of clay, and which, in the judgment of the Engineer, may be removed without blasting, although blasting may occasionally be resorted to.

SOLID ROCK will include all rock found in ledges, or masses of more than one (1) cubic yard, which, in the judgment of the Engineer, may be best removed by blasting, with the exception of stratified rocks described under the head of Loose Rock. In rock excavations the "bottom" must in all cases be taken down truly to sub-grade; and when so ordered by the Engineer ditches must be formed at the foot of the slope.

The contract price for excavations will apply to pits required for foundations of masonry when water is not encountered, and the price for

EXCAVATION IN WATER will only apply to foundation pits under water and deepening of channels in running water; it must cover all classes of material, and include drainage, bailing, pumping, and all materials and labor connected with such excavations, also the necessary dressing of the rock.

CEMENT must be of the best quality of freshly burned and ground hydraulic cement, and be equal in quality to the best brands of   cement. It will be subject to test made by the Engineer or his appointed inspector, and must stand a proof tensile test of fifty (50) pounds per square inch of sectional area on specimens allowed a set of thirty (30) minutes in air and twenty-four (24) hours under water.

MORTAR will in all cases be made of one part in bulk of the best hydraulic cement to two parts in bulk of clean, sharp sand, well and thoroughly mixed together in a clean box of boards, before the addition of the water, and must

be used immediately after being mixed. No mortar left overnight will, under any pretext, be allowed to be used. The sand and cement used will at all times be subject to inspection, test, acceptance, or rejection by the Engineer.

CONCRETE.—Concrete shall be composed of fragments of hard, sound and acceptable stone, broken to a size that will pass through a two (2″) inch ring in any direction, thoroughly clean and free from mud, dust, dirt, or any earthy admixture whatever; mixed in the proportion of two (2) parts in bulk of the broken stone to one (1) part of fresh-made cement mortar of the quality herein described; and is to be quickly laid in sections and in layers not exceeding nine (9) inches in thickness, and to be thoroughly rammed until the mortar flushes to the surface; it shall be allowed at least twelve (12) hours to "set" before any work is laid on it.

FOUNDATIONS

GENERAL DESCRIPTION.—Foundations for masonry shall be excavated to such depths as may be necessary to secure a solid bearing for the masonry, of which the Engineer shall be the judge. The materials excavated will be classified and paid for, as provided for in these specifications, under the general head of Excavations; and in case of foundations in rock, the rock must be excavated to such depth and in such form as may be required by the Engineer, and must be dressed level to receive the foundation course.

When a safe and solid foundation for masonry cannot be found at a reasonable depth (to be judged of by the Engineer), there will be prepared by the contractor such artificial foundations as the Engineer may direct. All materials taken from the excavations for foundations, if of proper quality, shall be deposited in the contiguous embankment; but any material unfit for such purpose shall be deposited outside the roadway, or in such place as the Engineer shall direct, and so that it shall not interfere with any drain or water-course.

TIMBER.—Timber foundations when required shall be such as the Engineer may by drawings or otherwise prescribe, and will be paid for by the one thousand feet, board measure. The price, covering cost of material, framing, and putting in place, and all wrought- and cast-iron work ordered by the Engineer, will be paid for per pound, the price including cost of material, manufacture, and placing in the work.

PILING.—All timber used in foundations or foundation piling shall be of young, sound, and thrifty white oak, yellow pine, or other timber equally good for the purpose, acceptable to the Engineer. Piles must be at least eight (8″) inches in diameter at the small end and twelve (12″) inches in diameter at the butt when sawn off; they must be perfectly straight and be trimmed close, and have the bark stripped off before they are driven. They must be driven into hard bottom until they do not move more than one-half inch under the blow of a hammer weighing two thousand (2000) pounds, falling twenty-five (25′) feet at the last blow. They must be driven vertically and at the regular distances apart from centers, transversely and longitudinally as required by the plans or directions of the Engineer; they must be cut off squarely at the butt and be well sharpened to a point, and when necessary, in the opinion of the Engineer, shall be shod with iron and the heads bound with iron hoops, of such dimensions as he may direct, which will be paid for the same as other iron work used in foundations.

The necessary length of piles shall be ascertained by driving test piles in different parts of the localities in which they are to be used; and in case a pile shall not prove long enough to reach " hard bottom " it shall be sawed off square, and a hole two (2″) inches in diameter be bored into its head twelve (12″) inches deep; into this hole a circular white-oak trenail twenty-three (23″) inches in length shall be well driven, and another pile similarly squared and bored, and of as large a diameter at the small end as can be procured, shall be placed upon the lower pile, brought to its proper position, and driven as before directed. All piles, when thus driven to the required depth, are to be cut off truly square and horizontal at the proper height given by the Engineer, and only the actual number of lineal feet of the piles left for use in the foundations after being sawn off will be paid for.

Coffer-dams.—Where coffer-dams are, in the opinion of the Engineer, required for foundations the prices provided in the contract for timber, piles, and iron in foundations will be allowed for the material and work on same, which is understood as covering all risks from high water or otherwise, draining, bailing, pumping, and all materials connected with the coffer-dams. Sheet-piling will be classed as plank in foundations, and will be paid for per one thousand (1000′) feet, board measure, if left in the ground.

TIMBER

All timber must be sound, straight-grained, and free from sap, loose or rotten knots, wind-shakes, or any other defect that would impair its strength or durability; it must be sawed (or hewed) perfectly straight and to exact dimensions, with full corners and square edges; all framing must be done in a thoroughly workmanlike manner, and both material and workmanskip will be subject to the inspection and acceptance of the Engineer.

APPENDIX IV

SPECIFICATIONS FOR STEEL COFFER-DAM

Design.—The shell shall be made of elliptical shape for ordinary piers and circular for pivot piers. It shall be made not less than four feet larger than footing of pier in plan, to allow for variation in position during sinking.

The plates used shall be as large as can be handled with ease in the shop, during shipment, and during erection.

The splices may be either lap or butt joints, provided a good tight job will result, and the rivets must be spaced according to boiler-maker's rules.

The joint may be made tight by calking or by the use of a calking strip, but in either event the result must be guaranteed.

The shell must be stiffened by horizontal stiffening angles, girders, or trussing, to resist deformation during the placing and to resist both the quiescent and a maximum unbalanced earth or water pressure, or a wind pressure.

The bottom plates shall be re-enforced with narrow plates inside and outside, to form a wedge-shaped cutting edge; and when there is rock or hard bottom, the plates shall be cut to conform to its contour as nearly as possible.

The top shall be properly stiffened, and if necessary provided with connection holes for additional sections.

The factor for safety shall in no case be less than four, and in case the shell will be subject to shock, not less than six.

No metal of a less thickness than $\frac{1}{4}$ inch shall be used for temporary work, nor less than $\frac{3}{8}$ inch for permanent work in fresh water or $\frac{1}{2}$ inch in salt water.

Material.—The entire shell shall be constructed of the grade of steel known as soft medium, except rivets, which shall be of bridge quality of iron.

The steel may be made either by the Bessemer or open-hearth process, and the phosphorus shall never exceed 0.08 per cent.

Soft medium steel shall have an ultimate strength of from 55,000 to 65,000 pounds per square inch, as determined from standard test pieces; an elastic limit of not less than one-half the ultimate strength; an elongation of not less than 25 per cent in 8 inches; and a reduction of area at fracture of not less than 50 per cent.

Samples to bend cold 180° to a diameter equal to the thickness of the sample, without crack or flaw on the outside of the bent portion.

Erection.—The erection must be done in a first-class manner, and all rivets must have full heads. The shell shall be placed in position within one-half the distance allowed for error in the design of the coffer-dam. Only a reasonable variation will be allowed for difference in level.

Painting.—All the metal work shall be thoroughly cleaned of rust or scale at the shops and coated thoroughly with hot asphaltum.

Before erection, in the field, it shall be given a second coating of hot asphaltum.

Sealing.—When in position on the bottom, if the coffer-dam has not been sunk through impervious strata, it shall be sealed by concreting around the circumference inside with concrete passed through a tube.

Removal.—Should the coffer-dam not form a part of the permanent foundation it shall be taken apart, at the joints designed for the purpose, and carefully removed in such a manner as not to injure the foundation, and so as to be used again if required.

APPENDIX V

SPECIFICATIONS FOR CEMENT SUBMITTED TO THE AMERICAN SOCIETY FOR TESTING MATERIALS*

Engineering News, June 30, 1904

IN our report last week of the annual meeting of the American Society for Testing Materials we stated that the Committee on Cement had submitted a report containing a recommended specification for natural and Portland cements, which specification was approved by the meeting and will now be voted on by the society at large, through letter-ballot. This specification is herewith presented in full. Its importance arises mainly from the fact that this society aims to produce specifications which shall be, as nearly as possible, standards for this country. The committees of the society are carefully selected from the most prominent representatives of the interests concerned and are intended to give equal hearing to manufacturer and consumer. The Committee on Cement was composed of thirty-one members, as follows:

George F. Swain, Professor of Civil Engineering, Mass. Inst. of Technology, Boston, Mass. (Chairman); George S. Webster, Chief Engineer, Bureau of Surveys, Department of Public Works, Philadelphia (Vice-Chairman); Richard L. Humphrey, Consulting Engineer and Chemist, Harrison Building, Philadelphia, Pa. (Secretary); Booth, Garrett & Blair, Engineers and Chemists, 406 Locust St., Philadelphia, Pa.; T. J. Brady, President, Coplay Cement Manufacturing Co., Coplay, Pa.; C. W. Boynton, Inspector of Cements, Baltimore & Ohio Railroad Co., Wheeling, W. Va.; Spencer Cosby, Major, Corps of Engineers, U. S. A., War Department, Washington, D. C.; Allan W. Dow, Inspector of Asphalts and Cement for the District of Columbia, Washington, D. C.; L. Henry Dumary, President, Helderberg Cement Co., Albany, N. Y.; A. V. Gerstell, General Manager, Alpha Portland Cement Co., Easton, Pa.; Edward H. Hager, Manager, Cement Department, Illinois Steel Co., Chicago, Ill.; Wm. H. Harding, President, Bonneville Portland Cement, Land Title Bldg., Philadelphia; Lathbury & Spackman, Engineers and Chemists, 1619 Filbert St., Philadelphia; Robert W. Lesley, President, American Cement Co., 22 So. 15th St., Philadelphia, Pa.; F. H. Lewis, Manager, Virginia Portland Cement Co., Fordwick, Va.; John B. Lohar, President, Vulcanite Portland Cement, Land Title Bldg., Philadelphia, Pa.; Andreas Lundteigen, Assistant Manager, Peerless Portland Cement

*As revised August 16, 1909.

Co., Union City, Mich.; W. W. Maclay, President Glens Falls Portland Cement Co., Glens Falls, N. Y.; Charles A. Matcham, Manager, Lehigh Portland Cement Co., Allentown, Pa.; Charles F. McKenna, Consulting Chemist, Laboratories, 221 Pearl St., New York; Spencer B. Newberry, Manager, Sandusky Portland Cement Co., Sandusky, Ohio; F. H. Bainbridge, Assistant Engineer of Bridges, Illinois Central Railroad Co., Chicago, Ill.; J. Madison Porter, Professor of Civil Engineering, Lafayette College, Easton, Pa.; Joseph T. Richards, Chief Engineer, Maintenance of Way, Pennsylvania Railroad Co., Broad St. Sta., Philadelphia, Pa.; Clifford Richardson, Director, New York Testing Laboratory, Long Island City, New York; Louis C. Sabin, Asst. Engineer, United States Engineer Office, Detroit, Mich.; Harry J. Seaman, Superintendent, Atlas Cement Co., Northampton, Pa.; S. S. Voorhees, Engineer of Tests, Supervising Architect's Office, Treasury Department, Washington, D. C.; Olaf Hoff, Engineer of Structures, New York Central & Hudson River Railroad, Grand Central Sta., New York; W. S. Eames, President, American Institute of Architects, St. Louis, Mo.; W. J. Kelly, Vice-President, American Railway Engineering and Maintenance of Way Association, Minneapolis, Minn.

The specifications drawn up by this committee are prefaced by some explanatory remarks and suggestions in regard to the various tests. For further explanations of the procedure of testing the committee refers to the report of the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, whose recommendations it adopts as standard and reprints as an appendix to its report. We have omitted this appendix and give below only the specifications and the prefatory remarks:

GENERAL OBSERVATIONS

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

3. Specific Gravity.—Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

4. Fineness.—The sieves should be kept thoroughly dry.

5. Time of Setting.—Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities vitally affect the rate of setting.

6. Constancy of Volume.—The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

7. In making the pats the greatest care should be exercised to avoid initial strains due to molding or to too rapid drying out during the first twenty-four

hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

8. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement may, however, be held for twenty-eight days, and a retest made at the end of that period, using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

STANDARD SPECIFICATIONS FOR CEMENT

1. GENERAL CONDITIONS.—All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the contractor and a period of at least twelve days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages with the brand and name of manufacture plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain four bags, and each barrel of natural cement shall contain three bags of the above net weight.

8. Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight-day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the society Jan. 21, 1903, amended Jan. 20, 1904, and Jan. 15, 1908, with all subsequent amendments thereto.

10. The acceptance or rejection shall be based on the following requirements:

NATURAL CEMENT

11. DEFINITION.—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

12. FINENESS.—It shall leave by weight a residue of not more than 10 per cent. on the No. 100, and 30 per cent. on the No. 200 sieve.

13. TIME OF SETTING.—It shall develop initial set in not less than ten minutes, and it shall not develop hard set in not less than thirty minutes, nor more than three hours.

14. TENSILE STRENGTH.—The minimum requirements for tensile strength for briquettes 1 square inch in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Neat cement:

Age.	Strength.
24 hours in moist air	75 lbs.
7 days (1 day in moist air, 6 days in water)	150 "
28 " (1 " " 27 ")	250 "
One part cement, three parts standard Ottawa sand:	
7 days (1 day in moist air, 6 days in water)	50 lbs.
28 " (1 " " 27 " ")	125 "

15. Constancy of Volume.—Pats of neat cement about 3 inches in diameter, one-half inch thick at center, tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° F. as practicable.

16. These pats are observed at intervals for at least twenty-eight days, and, to satisfactorily pass the tests, should remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

Portland Cement

17. Definition.—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent. has been made subsequent to calcination.

18. Specific Gravity.—The specific gravity of cement shall be not less than 3.10.

Should the test of cement as received fall below this requirement, a second test may be made upon a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4 per cent.

19. Fineness.—It shall leave by weight a residue of not more than 8 per cent on the No. 100, and not more than 25 per cent. on the No. 200 sieve.

20. Time of Setting.—It shall develop initial set in not less than thirty minutes, and must develop hard set in not less than one hour, nor more than ten hours.

21. Tensile Strength.—The minimum requirements for tensile strength for briquettes one square inch in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified·

Neat cement:

Age.	Strength.
24 hours in moist air	175 lbs.
7 days (1 day in moist air, 6 days in water)	500 "
28 " (1 " " 27 " ")	600 "
One part cement, three parts standard Ottawa sand.	
7 days (1 day in moist air, 6 days in water)	200 "
28 " (1 " " 27 ")	275 "

22. Constancy of Volume.—Pats of neat cement about 3 inches in diameter one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least twenty-eight days.

(*b*) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least twenty-eight days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

23. These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegration.

24. Sulphuric Acid and Magnesia.—The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

APPENDIX VI

METAL SHEET-PILING

THE prediction was made, in the first edition of this book, at the end of Chapter VI, that Metal Sheet-piling would doubtless come into use as timber became more expensive to use, and at the end of Chapter VIII mention is made of work at Cuxhaven Harbor, Germany, where Metal Sheet-piles were used. An account is also given of the Friestedt Patent Interlocking Sheet-piling, which is almost identical with Metal Sheet-piling described in Volume I of the "Transactions of the Institution of Civil Engineers." It is worthy of comment that this has probably been lost sight of by engineers, and even "The Engineer" in a review of the first edition of this book makes the statement:

"Numerous existing examples of coffer-dams constructed of sheet-piling are described and illustrated, and Mr. Fowler endorses a statement recently made in our columns by remarking that "the growing scarcity of timber will doubtless lead to the use of metal at some time in the future to replace sheet-piling for coffer-dams." So that the following paper on Metal Sheet-piling, published in 1836, will doubtless prove of great interest to engineers, if not of considerable value:

"Memoir on the use of Cast Iron in Piling, particularly at Brunswick Wharf, Blackwall. By Michael A. Borthwick, A. Inst. C.E."

A short sketch of the introduction and use of cast iron in piling may not be considered an inappropriate accompaniment to an account of one of the most recent works in which it has been adopted.

Public attention was first drawn to such an application of iron by Mr. Ewart, of Manchester, now of His Majesty's Dock-yard, at Woolwich; but though this merit is certainly due to that ingenious gentleman, he had been, as it afterwards proved, anticipated in the idea by the late Mr. Mathews, of Bridlington, who, previously to the date of Mr. Ewart's patent, had used cast-iron sheet-piles in the foundations of the head of the north pier of that harbor. These piles were of different forms; in the margin (Fig. 471) is given a cross-section of one of, I believe, the most common, in which it will be seen the adjoining piles dovetail to each other, while in others, I have been informed, they merely overlap. Their length was about 8 or 9 feet, their width from 21 inches to 2 feet, and their thickness half an inch.

In ignorance of Mr. Mathews' proceedings, Mr. Ewart, in the beginning of 1822, took out a patent for a new method of making coffer-dams, which he proposed to effect by employing plates of cast iron, held together by cramps fitted to dovetailed edges on the piles. A section of these piles, taken from some that have been used, is shown in the accompanying sketch (Fig. 472).

A detail of the mode in which it was proposed to combine them so as to form a coffer-dam might be out of place, in a paper that has reference more to the use of iron piling for permanent purposes; the plan, as described in the specification of the patent, is to be found in the Repertory of Arts, and an abstract of it in the *London Journal of Arts and Sciences* for the year 1822. The length of the piles is therein stated as intended to be from 10 to 15 feet, which is, I understand, about what they have generally been made, and for cases requiring a greater depth, a mode is described of lengthening the piles, by placing one above another, and securing the horizontal joints by means of dovetailed cramps.

Though, on being apprised of what had been done at Burlington, Mr. Ewart did not defend his patent, his piles have been pretty extensively adopted, particularly by Mr. Mylne, of New River Head, London, and Mr. Hartley, of Liverpool. Besides other operations in the important public work under his charge, the former gentlemen used the piles, soon after their invention, with complete success in a coffer-dam of considerable size, constructed in the river Thames for the purpose of putting in a suction-pipe opposite the New River Company's establishment at Broken Wharf. They have also been used with advantage by Mr. Hartley, in founding the pier heads of the basin of George's Dock, and various parts of the walls of some of the other docks at Liverpool, as also in putting in the foundations of the south river-wall.

FIG. 471.—MATTHEWS' CAST-IRON SHEET-PILE.

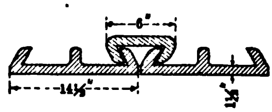

FIG. 472.—EWART'S CAST-IRON SHEET-PILE.

Looking at the dovetailed form of these piles, one would, I think, have been inclined to anticipate difficulty in driving them, but this does not seem to have been met with to any extent in practice, at least in coffer-dams, the original object of the invention. On this point I have pleasure in being able to quote some observations of Mr. John B. Hartley, which contain the results of the Liverpool experience: "Considerable care," he writes, "is required in keeping the piles in a vertical position, as they are apt to shrink every blow and drive slanting. They require to be driven between two heavy balks of timber to keep them in a straight line, as they expose very little section to the blow of the ram, and are so sharp that they are easily driven out of a right line. There is another very necessary precaution to be taken, which is the keeping of the fall in the same line as the pile; otherwise the ram descending on the pile and not striking it fairly, all parts equally, the chances are that, if in a pretty stiff stratum, the head breaks off in shivers, and the pile must be drawn, which is sometimes no easy matter." He concludes by saying, "these piles are on the whole the most useful tools you can use for their purpose (coffer-damming). I believe they have had as extensive a trial at the Liverpool Docks as anywhere else, and certainly with success. They have generally been driven with the ringing or hand engine and rams of 3 or 4 cwt., a front and back pile being driven at the same time by one ram."

In the work at Broken Wharf, the practice was to insert the piles and cramps

all round the dam first, and drive them a moderate distance into the ground, then to pass the engine repeatedly round and send them down gradually, instead of driving them home at once; and Mr. Mylne has mentioned to me that while this was in progress, the piles being at the time but slightly driven, he was somewhat alarmed one morning at finding that the run of the water had elevated one end of the dam considerably above the other. The dovetails, however, held good, and proper precautions being taken, the return of the tide put all right again without at all crippling the work, the movement having been regular all over the dam. I ought to add that these dams are still used in the works on the New River, four sets being generally kept in hand, and that the ringing engine is always employed, and the above stated method of driving followed.

I have perhaps dwelt longer on Mr. Ewart's project than I should otherwise have done, from a feeling that from his labors has sprung much that has followed in the way of iron piling; and besides, it may be observed, the remarks as to driving are not entirely limited in their application to this particular description of pile. The next work that occurs was executed by Mr. Walker in 1824; this was the rebuilding of the return end of the quay-wall of Downes Wharf, Saint Katherine's, which had been undermined by the wash from the Hermitage entrance of the London Docks. With a view to a more effectual resistance of a like action in future, iron instead of wood sheet-piling was introduced in the foundation of the wall in question; and though, if one may judge from the specification of the patent, no application of his plan of so permanent a nature seems to have been contemplated by Mr. Ewart, the work was begun according to it, but it was afterwards modified at the request of the contractor, so as to give the section of pile shown in the margin (Fig. 473), the flanch being in front or outside. Although, as has been already seen, the piles in their original form may be easily enough driven in some cases, it was found impossible to get them down in a regular line to the depth required in the present instance, through the hard material that had to be penetrated, and by which in fact they were surrounded and pressed for nearly their whole length of 14 feet.

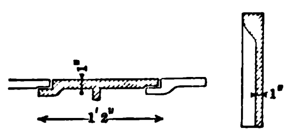

FIG. 473.—EWART'S MODIFIED SHEET-PILE.

A work on a much larger scale than any yet mentioned now presents itself, the wharfing at the sea entrance of the Norwich and Lowestoft navigation. In this Mr. Cubitt has adopted sheet-piling exclusively without the intervention of main or guide piles; the form and section will be seen by the accompanying sketches (Fig. 474), which it is almost unnecessary to observe are not drawn to the same scale, the transverse section being considerably enlarged beyond the other two. The piles are all 30 feet long; their weight is about a ton and a half each. The back flanch, which is shown at the deepest on the cross-section, tapers gradually to about 6 inches at top, where the angles are blocked in to form a head for driving, and is diminished at the lower end by steps or set-offs of parallel width with square ends, instead of a straight or curving line, as the latter shape was found to have a tendency to press the pile forward, whereas by the plan adopted it drove as fairly as if the flanch had been continued its full width to the foot of the pile. The driving was all effected by means of crab engines with monkeys about as heavy as the piles, no more fall being allowed than was necessary to send them

down, and the whole is secured by land ties, two in height, at intervals of six feet. The entire length of wharfing thus constructed is about 2000 feet.

From the form of the pile, according to this plan, giving so thin an abutting surface, and the joints not being covered in any way, close and accurate driving seems essential to its efficacy, and the nature of the ground (sand mixed with shingle) would have made this a somewhat troublesome operation at Lowestoft, but for the plan that was taken to insure precision. This consisted in riveting close to the lower end of the pile about to be driven a pair of strong wrought-iron cheeks projecting beyond the edge about 2 or 3 inches, which clasping the pile already driven, served as a guide or groove to keep the piles flush, however this the edge * and the tendency to turn out or in at the heel was counteracted after a few trials by giving a greater or less bevel to the front or back face. With these appliances the piling was pretty closely driven, and the work, which was completed in 1832, has been found fully to answer the object of supporting the sides of the cut from Lake Lothing to the sea against the effects of the very ingenious and powerful sluicing apparatus provided in the lock at that place.

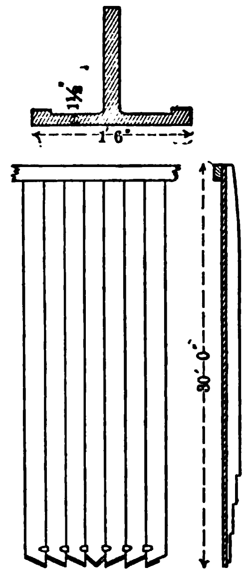

FIG. 474.—CUBITT'S IRON SHEET-PILING.

About a year later than the above, Mr. Sibley constructed an iron wharfing on the Lea Cut at Limehouse on an opposite principle, sheet-piling being in this case altogether discarded, and the work consisting of flat plates let down in grooves on the sides of guide-piles of an elliptical form according to the section opposite (Fig. 475), driven at distances of 10 feet. These piles are 20 feet long, weigh about 1½ tons each, and are 9 feet apart; they are hollow throughout, to enable a passage for them to be bored in the soil by means of an auger passed through them, and so ease the driving and are filled with concrete; each pile is land-tied, and the plates extend to within 6 feet of the point. A similar wharfing, but on a larger scale, has since been made on each side of the Thames, adjoining New London Bridge; that on the city side rather an extensive work, the piles in it being 43 feet long (cast in two unequal lengths with a spigot and faucet joint), of a cylindrical form, 12 inches diameter, and of metal 1½ inches thick, and each pile being secured by two tiers of ties of 2-inch square iron carried 70 or 80 feet back, to resist the great depth of filling up or backing.

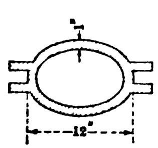

FIG. 475.—SIBLEY IRON SHEET-PILING.

The plan just described seems well enough adapted for situations where any great increase of depth is not likely to take place. The absolute depth is not so important, though where this is considerable, it may be questionable whether a heavy wharf would not be the better for the protection of a continuous row of piling at foot; the strong land-tying necessary in the last-mentioned work seems to point to this.

* This plan has, I believe, been followed by Mr. Cubitt in driving timber-piling also, in cases requiring nicety of work.

I now come to the quay-wall constructed in 1833–34 by Messrs. Walker and Burges on the river Thames, in front of the East India Docks at Blackwall and since named Brunswick Wharf. The object of this work was to afford accommodation for the largest class of steam-vessels at all times of tide, for which the old quay, even had it not been in a state of decay, was not adapted from the shallowness of the water in front of it. To effect this, the first idea was to run out two or three jetties from the wharf, but this was soon abandoned, and a new river-wall resolved on; and advantage was taken of the occasion to improve the line of frontage by an extension into the river, under the sanction of the Navigation Committee of the Port of London, varying from a point at the east end to about 25 feet at the other extremity. The use of iron in the work was, I have understood, suggested by Mr. Cotton, deputy chairman at the time, and for many years an active member of the most respectable and liberal body then in the direction of the East India Dock Company, and the adoption of the proposal was facilitated by the circumstance which probably led in the first instance to its being made, namely, the low price of the material at the period, the contract being little more than 7 pounds per ton delivered in the Thames.

In the accompanying drawing (Fig. 476) an attempt is made to show the mode of construction that was followed, so as to avoid the necessity for much written detail. The first operation was to dig a trench 2 yards deep in the intended line, and this was immediately followed by the driving of the timber guide-piles. The deepening in front, which, to give the required depth of 10 feet at low water, was as much as 12 feet, was not done until near the conclusion of the work; to have effected it in the first instance would without any countervailing advantage, except some saving in the driving, have been attended with the double expense of removing the ground forming the original bottom between the old and new lines of wharfing, and afterwards refilling the void so left by a material that would require time to make it of equal solidity; and even if this had been otherwise, such an attempt would have endangered the old wall, or rather would have been fatal to it. The permanent piling was next begun, the main piles being driven first at intervals of 7 feet, and the intermediate spaces or bays then filled in, working always from right to left, towards which the drafts of the sheet-piles were pointed. The ground is a coarse gravel, with a stratum of the hard Blackwall rock occurring in places, and some trouble was occasionally experienced from its tendency to turn the piles from the proper direction, but, due attention being paid to the form of the points, the driving was on the whole effected pretty regularly, but few of the bays requiring closing piles specially made for them, so that the work may be said to be nearly iron and iron from end to end; at the same time, the vertical joints of the piling being all covered, as will be noticed presently, any slight imperfection in this respect is no serious detriment to the work as a whole.

The main piles are in two pieces, the lower end of the upper one being formed so as to fit into a socket on the top of the under length, and the joining made good by means of a strong screw-bolt; the only object of this was to insure a supply of truer castings, and lessen the difficulty of transporting such unwieldy masses from Northumberland and Staffordshire to London.* Each sheet-pile is secured at the top by two bolts to the uppermost wale of the woodwork behind, and the edge of the end ones of each bay, it will be observed, passes behind the

* The Birtley Iron Company, Newcastle-on-Tyne, were the contractors for the ironwork but a portion was supplied by the Horsely Iron Company. Mr. M'Intosh, of Bloomsbury Square, had the contract for driving the piles and fixing the work.

adjoining main pile, while the other joints are overlapped by the bosses with which all the sheet-piles except the closers are furnished on one side. Besides adding to the perfection and security of the work by breaking the joints, so that

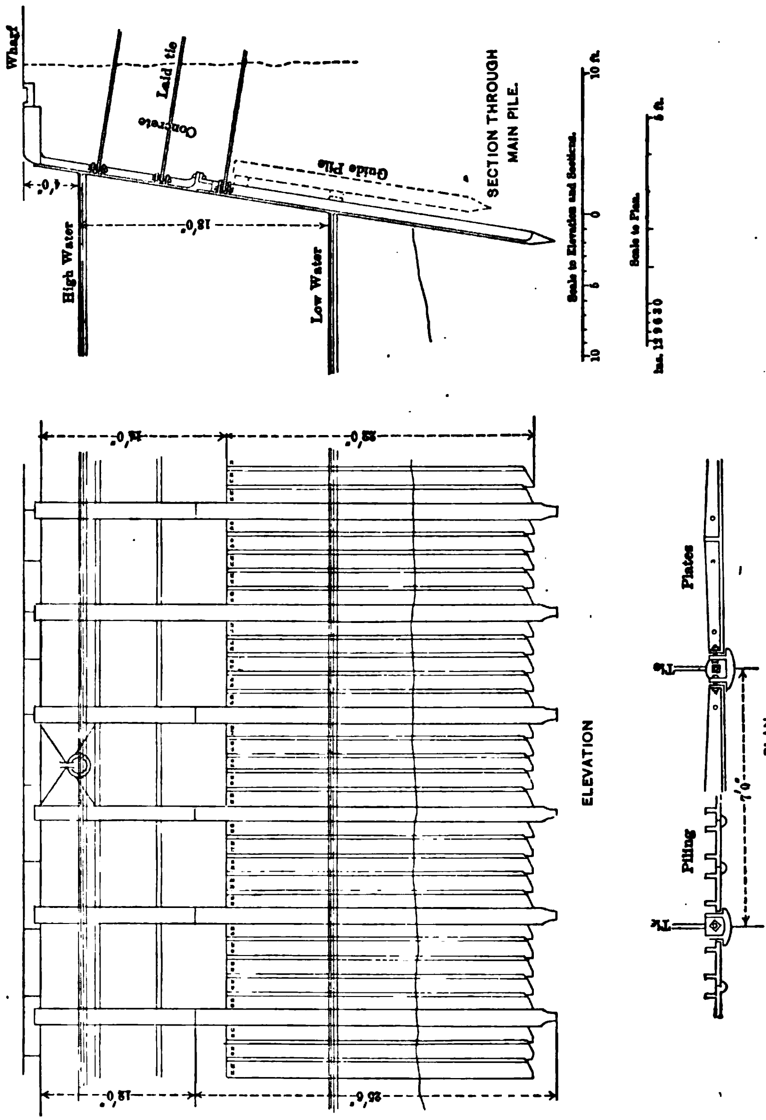

FIG. 476.—BRUNSWICK WHARF, IRON SHEET-PILING.

the water (if it penetrate, as with even the best pile-driving it will) cannot draw the backing from its place, these projections appear to me to relieve the appearance of the otherwise too uniform face; and a like effect is produced by the horizontal fillets on the lower edges of the plates above, which also mask the joints. These

plates, filling up the spaces over the sheet-piling, are bolted to the main piles and to each other in the manner shown, and the joints stopped with iron cement. Where the mooring-rings come, the plates are cast concave, with a hole perforated in the middle to allow a bolt to pass through, and this bolt is secured, as well as the land-ties from the main piles, to the old wharf, which was not otherwise disturbed, or to needle-piles driven adjoining it. The backing consists of a concrete of lime and gravel, in the proportion of about one to ten, extending down to the solid bottom. The coping with the water channel in its rear is of Devonshire granite; the water is conveyed from the channel at intervals by pipes, extending from gratings in the bottom in a slanting line to the lowermost plate, discharging themselves immediately above the sheet-piling.

The main piles were originally proposed to be hollow in section, according to the sketch opposite (Fig. 477); but this was given up on further consideration of the uncertainty of procuring sound castings of the intended form, and of the greater liability to break afterwards from a blow sidewise. The solid form shown on the plate was therefore adopted, according to which the lower lengths weighed about 28 cwt.; and that this was not too much was shown by the circumstance of several of the piles, particularly the early ones, breaking in the testing or driving, and showing in the fracture the danger of even a slight defect. The greater care subsequently taken at the foundry, and probably also greater experience in driving, made accidents of this kind of rarer occurrence in the later stages of the work; and it may be mentioned as no bad proof of the care of all parties, that of upwards of six hundred piles, including both descriptions, only sixteen broke in driving, seven being of one sort, and nine of the other; the failure was in five cases attributed to strains in driving, and to imperfections of casting in the other eleven. The sheet-piles, which bear a considerable resemblance in their general outline to those used at Downes Wharf ten years before, were proposed to be an inch thick, but it was found necessary to increase this dimension, and some of them were as much as 1¼ inches; the average, however, was not above 1⅛ inches, and the weight of each pile 17 cwt. The length of the wharf is about 720 feet, and the whole weight of iron used upwards of 900 tons.

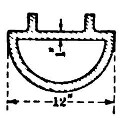

Fig. 477. — Original Pile Proposed for Brunswick.

The crab engine was employed invariably, the heads of the piles being covered with a slip of ¾-inch elm, to distribute the force of the blow equally over the iron, and prevent jarring. The monkeys used weighed from 13 to 15 cwt. each, and it was found necessary to limit the fall to a height of 3 feet 6 inches, and sometimes less, when the resistance proved more than usually great and the pile showed a tendency to turn from its straightforward course. The driving throughout was very hard, more especially at the west end, where the sheet-piles in four bays could not be forced to the full depth, the space above being in two of them made up with two plates in height, and in the other two admitting only one, instead of three as in the rest of the work. Driving was the only means resorted to, or indeed practicable in the gravelly soil that prevailed. Had the bottom been clay or other similar substance, the plan of boring to receive the points, that has been followed elsewhere, might probably have been partially adopted in the main piles with advantage; but I should say, certainly not to the extent of depending mainly upon it for getting the piles home to their places.

I cannot quit the subject of the Brunswick Wharf without stating that his avocations alone have prevented Mr. George Bidder's association with me in the account of a work, the execution of which he had, under Messrs. Walker and Burges, the charge of superintending. Though rejoicing at the cause, I cannot help regretting the circumstance in the present instance, as such co-operation on the part of my friend would, I feel, have given this paper an interest and a value it has now but little claim to. I take this opportunity also of acknowledging my obligation to several of the gentlemen above named in connection with the previous use of iron piling, whose kindness has enabled me to make the preliminary review much fuller than I had at one time any expectation of having the power to do.

It remains for me only, in conclusion, to advert to a consideration that ought not to be lost sight of in deciding upon the eligibility of cast-iron wharfing—I mean the action of water upon it. I do not recollect any observations made so as to enable a practical inference to be drawn from them; but the importance of the subject seems to claim attention, and possibly even this notice may be the means of inducing it from those who have the opportunity.

The investigation belongs perhaps rather to chemistry than engineering, but notwithstanding the practical turn some of the most distinguished cultivators of that science have given their researches, little I believe has yet been done to explain the present question. How iron is affected by water in its various states, and in what manner the action on wrought differs from that on cast iron, are interesting points, still, so far as my information goes, to be determined; and they are not likely, to be so in a satisfactory manner until some one competent to the task calls a series of well-conducted experiments in aid, as every day shows more clearly the uncertainty of analogical reasoning, however appararently strict, on such subjects. But whatever the modus operandi between cause and effect, that decomposition of the metal, more or less rapid, gradually goes on from the action of water, seems to admit of no doubt. Professor Faraday, in a letter to Captain Brown, says," Cast iron is certainly liable to great injury from constant immersion in salt water, and I think you would find few, if any exceptions, provided the water and the iron are in contact."* And the saline principle, to use a somewhat antiquated form of expression, though a great accelerator of the process, does not appear to be altogether an essential to it;† at least, I know a case that happened in a part of the river Thames where the water cannot be said to be more than brackish at any time, and indeed is generally quite fresh, in which cast iron, after being immersed for little more than twenty years, was, on being withdrawn from the water, found so soft as to yield to the penknife; and the original surface of the iron referred to—it was the socket-plate to the heel-post of a lock-gate—had not been submitted to the tool, in which case it is well known the water would have operated with much greater effect.

But though I have thought it well to glance at the above case occurring in water, always except on rare occasions fresh, the sea is no doubt in practice, the invader whose inroads are most alarming. Instances might easily be cited in proof of the ravages committed by that active enemy, though not perhaps noted so circumstantially as is desirable, but I am unwilling to lengthen this communication further, and shall therefore confine myself to a passing allusion

* Description of a Bronze or Cast-iron Columnal Lighthouse, etc., by Capt. Brown, R.N.

† The difference between sea and other water, in operating with the galvanic battery, is much less considerable than that between the latter and distilled, but it is between salt and fresh that the practical question lies in the present case.

to the example on a large scale, and after long trial, furnished by the state of the guns taken from the wreck of the *Royal George*, as described at a late meeting of the Institution;* and to a similar instance mentioned by Berzelius, in a passage which I quote at length, not so much however in confirmation of so well established a fact as the eventual decomposition of cast iron by the action of water, as for the properties mentioned of the substance into which the metal is resolved. The extract is as follows:

"Quand la fonte reste long-temps sous l'eau, elle est decomposée; l'acide carbonique contenu dans l'eau dissout le fer et l'entraine; il reste une masse grise qui ressemble à la plombagine. Lorsqu'on retira de l'eau, il y a quelques annés, les canons d'un vaisseau qui avait coulé à fond cinquante ans auparavant, aux environs de Carlscrona, on les trouva au tiers converti en une pareille masse poreuse; à peine étaient-ils à l'air depuis un quart d'heure, qu'ils commencèrent à s'échauffer tellement, que l'eau qui y restait encore s'échappa sous forme de vapeur, et qu'il fut impossible d'y toucher. Depuis, Macculloch a observé † que le corps analogue à la plombagine qui se forme ainsi présente toujours ce phénomène, et que ce corps s'échauffe presque jusqu'au rouge, en absorbant de l'oxygène. Ou ne sait pas précisement ce qui se passe dans ce cas."—Traité de Chimie, Tom, III, p. 273.

* Min. of Convers. Vol. V., No. 12.

† The observation referred to by Berzelius in the above occurs in Macculloch's Western Isles of Scotland (I think in the account of the island of Mull), where an explanation of the phenomenon was first attempted, though, if on such a subject I may "hint a doubt," not to my mind quite a satisfactory one. A more perfect solution will probably be furnished by whoever, availing himself of the powerful means of chemical analysis now possessed, may undertake such an investigation of the whole question of the action of water on iron as I have ventured to allude to in the text.

APPENDIX VII

SPECIFICATIONS FOR FLOATING PILE DRIVER U. S. ENGINEERS, COLUMBIA RIVER, OREGON

DETAILED SPECIFICATIONS

General Description.—The work to be done under these specifications includes the building of a scow barge; erecting thereon a set of gins and a house; and furnishing and installing all the machinery hereinafter specified. All patterns required for the hammer, sheaves, bearings, etc., shall become the property of the United States and shall be delivered with the driver.

Material and Workmanship.—All lumber shall be of the best quality close-grain yellow fir, except where otherwise specified, without shakes, rot splits, unsound or large knots, sap or other imperfections. Decking and planking shall be edge grain, free from knots on face and edges, thoroughly seasoned and dry, but not kiln dried. All other timber shall be as well seasoned as time and circumstances will permit. All lumber shall be surfaced on four sides, the sizes given being for the timber in the rough.

All fastenings shall be galvanized. All bolts shall have washers under the nuts. Holes for fastenings shall be bored, allowing $\frac{1}{16}''$ for drifting.

All valves shall be "Lunkenheimer," "Powell," or equal; those 3″ and under shall be all brass and those above 3″ shall be cast iron with brass mountings.

All other materials shall be the best of their respective kinds and all work shall be done in a workmanlike manner.

Dimensions.—Length over all, 70′; width, 24′; moulded depth, 4′; crown of deck, 4″; height of gins, 66′.

Gunwales.—Each gunwale shall be built up of 5 strakes of the following dimensions: the bottom strake, 12″×12″ in one length; the second strake, 6″×12″ in two lengths; the third strake, 6″×12″ in three lengths; the fourth strake, 6″×12″ in two lengths, and the fifth strake, 6″×6″ in one length. All joints shall be 4′ scarfed joints, well broken so that no joint is directly over another.

The gunwale timbers shall be well fitted and fastened to a 6″×6″ rake timber and an 8″ knee as shown in Fig. 44. At the stern they shall be dovetailed into the transom as shown.

The gunwale strakes shall be fastened together with $\frac{3}{4}''$ drift-bolts and clinch-bolts, driven with $\frac{1}{16}''$ drift. A complete set of drift-bolts shall be driven for each strake as it is put up. They shall be spaced 2′ centers and each bolt shall go through two and one-half timbers, except the bottom set, which shall go through the second strake and within one inch of bottom of bottom strake. The clinch

bolts shall be spaced 8′ centers and extend through all strakes and be clinched over rings.

The strakes shall be closely joined together. The sides, scarfs and joints shall have calking seams $\frac{3}{16}''\times 3''$.

TRANSOMS.—The forward transom shall be built up of three $8''\times 12''$ timbers properly shaped for the crown of the deck and rake of plank.

The after transom shall be built up of 5 strakes of the following dimensions: the bottom strake, $12''\times 12''$; the next three, $6''\times 12''$, and the fifth, $6''\times 10''$. They shall be properly shaped for the crown of deck and rabbeted for planking. All transom timbers shall be fitted and fastened the same as specified for gunwales. At each dovetailed corner there shall be one $\frac{3}{4}''$ bolt clinched over rings.

BULKHEADS.—There shall be two longitudinal bulkheads, built up of one $8''\times 8''$ strake in one length, one $6''\times 10''$ strake in six lengths, two $6''\times 10''$ strakes in three lengths with scarfed joints, and one $6''\times 10''$ strake in one length, all drift-bolted and through fastened the same as specified for gunwales. The bulkheads shall be secured to the after transom by $8''\times 8''$ posts and to the forward transom by 8″ knees as shown, all through bolted as directed.

TRUSSED STRINGERS.—There shall be five trussed stringers built up as shown in Fig. 47, with the moulded depth to suit the crown of deck. The bottom member shall be $8''\times 8''$ in one length; the top member, $6''\times 6''$ with joints as shown. The posts shall be $6''\times 6''$ and the diagonals $3''\times 4''$, all well fitted with close joints. At each end of each stringer there shall be one 8″ knee connection to the transoms fastened with seven $\frac{3}{4}''$ through bolts. At every other post there shall be one $\frac{3}{4}''$ bolt passing through both members and clinched over rings. The other fastenings shall be $\frac{1}{2}''$ and $\frac{3}{4}''$ bolts, placed as directed. Limbers $1\frac{1}{2}''\times 3''$ shall be cut as directed.

CLAMP STRINGERS.—The clamp stringers shall be $4''\times 6''$ secured to the gunwales by $\frac{5}{8}''$ carriage bolts spaced one foot apart, heads outside let into counterbored and plugged holes.

CROSS BEAMS.—There shall be five $8''\times 8''$ cross beams placed as shown with an 8″ knee at each end of each beam. The posts and diagonal braces shall be as shown, well fitted and fastened. There shall be seven $\frac{3}{4}''$ through bolts through each knee placed as directed, and a $\frac{3}{4}''$ drift-bolt at every crossing of beam and bottom stringers.

DECK BEAMS.—There shall be five $8''\times 6''$ deck beams; the others shall be $4''\times 6''$ spaced 2′ centers. The 8″ beams shall be fastened at each trussed stringer by two $\frac{5}{8}''$ carriage bolts, and at each end and at each bulkhead by two $\frac{3}{4}''$ drift-bolts. The 4″ beams shall be fastened at each end and at each trussed stringer by one $\frac{5}{8}''$ carriage bolt and at each bulkhead by one $\frac{5}{8}''$ drift-bolt.

STANCHIONS.—There shall be four $8''\times 8''$ stanchions connecting the after transom to the gunwales and bulkheads. There shall be six $8''\times 8''$ stanchions or tow posts arranged as shown in Figs. 44 and 45, with the corners of the upper part neatly mitered. All stanchions shall be securely bolted as directed with $\frac{3}{4}''$ carriage bolts, heads outside let into counterbored and plugged holes.

BOTTOM PLANKING.—The bottom planking shall be $4''\times 10''$ run athwartship as shown. The edges shall be slightly beveled to give sufficient calking seams, and the planks laid with close joints inside. Each plank shall be fastened at each crossing by three 8″ ship spikes. The heads shall be let into counterbores and plugged with wooden plugs dipped in white lead.

DECK PLANKING.—The deck planking shall be $3\frac{1}{2}''\times 6''$ in lengths of not

less than 32′ with butts well shifted; no two butts shall be on the same beam unless at least three planks intervene. Each plank shall be spiked at each crossing by two 7″ ship spikes; the heads shall be let into counterbored and plugged holes.

GUARDS.—The upper guard shall be 3″×12″ on sides and 3″×16″ across the ends; the lower guard, 3″×8″ continued along the bow and joined to the upper guard. At the after end, 3″×8″ vertical guards shall join the lower to the upper guard. All guards shall be well spiked with 8″ ship spikes and the heads shall be let into counterbored and plugged holes. $\frac{3}{8}$″×6″×6′ corner irons shall be placed at each corner, both top and bottom, and fastened with eight 8″ countersunk spikes.

HATCHES.—There shall be three 2′×3′ hatches at each end, located as directed. Each shall be provided with both a removable lattice cover and a solid cover, the latter being flush with deck and provided with iron lifting rings.

CALKING.—All seams in bottom, gunwales, transoms, and deck shall be calked with three threads of best oakum, each thread well driven. All seams below the water line shall be payed with white lead and filled with a good grade of Portland cement troweled down smooth. The deck and all other seams above water line shall be payed over oakum with a good grade of pitch.

HOUSE.—A house 16′×30′6″ shall be built substantially as shown on the drawings. The coaming shall be 6″×6″ shaped to suit deck, the studding 4″×4″ spaced about 3′ centers or to conform to doors and windows, and mortised into coaming. The plate shall be 4″×6″ and the nailing strips, 2″×4″. All timbers shall be well framed and nailed as directed. There shall be one $\frac{3}{4}$″ tie-rod at each corner and two on each side, running through the carlins and deck beams or filling timbers. The carlins shall be 3″×12″ spaced 3′ centers and sawed with a 6″ camber. They shall be bolted at each end into the plate by a $\frac{1}{2}$″ carriage bolt.

The carlins shall be covered with $1\frac{1}{4}$″×6″ T. & G. material, nailed at every crossing, over which shall be laid athwartship, in paint, No. 5, 22″ cotton duck. Laps shall be fastened with double pointed galvanized tacks driven diagonally. A water course shall be run as shown, leading water into lead and galvanized iron scupper pipes one located at each corner. The siding shall be $1\frac{1}{4}$″×6″ T. &.G material run vertically and nailed at every crossing with 8d. wire nails.

The after end shall be provided with a 16-oz. canvas hood, hung by brass hooks and eyes, made so as to fasten down and cover all parts of the engine outside of the house.

The doors shall be constructed as shown, the side and forward end doors hung on Richards No. 28, or equal, hangers and track, and the after end doors hung on Richards No. 235, size 1, or equal, swivel trolley hangers and track. The doors shall be fitted with heavy hasps and staples arranged for padlocks. All windows shall have $1\frac{3}{8}$″ sash glazed with 26-oz. crystal glass. They shall be fitted to drop into ceiled pockets in the usual manner.

A work bench shall be constructed in a substantial manner of the dimensions shown. It shall have drawers and doors arranged as directed. A suitable tool board shall be installed on the wall near the bench.

GIN SILLS.—There shall be two longitudinal gin sills each 12″×12″×50′ long with drift-bolts every two feet extending at least 16″ into solid bulkheads. In addition there shall be two $\frac{3}{4}$″ bolts at each end and two at the back brace connections with nuts in pockets in the bulkheads. The machinery founda-

tion bolts shall be arranged in the same manner. There shall be one cross sill 12"×12" in three lengths, fitted into longitudinal sills and corner stanchions, well drift-bolted and through bolted into transom, stanchions, and sills by ¾" bolts.

GINS.—The gin timbers shall be 8½"×12" by approximately 66′ long, in one length. Each shall be notched into the sill timber as shown, and bolted by five ¾" carriage bolts at the bottom and at the top as shown in detail in Fig. 44. Each gin shall be beveled to fit and be faced with an 8"×11.25 lb. channel iron, fastened with ½" countersunk head-bolts spaced 1′ centers staggered. The lower part of channels shall be in one length at least 55′ long, the splice to the upper part being madé with a ⅜"×7" plate, well riveted.

BACK BRACES.—There shall be two back braces each 5"×12". Each brace shall have the lower end through bolted into sill timber by five ¾" bolts and the upper end framed and fastened as shown in detail in Fig. 44. The two braces shall be joined together by 2"×6" material, spaced 15" centers, forming a ladder. Each piece shall be fastened by four 4"×¼" ship spikes.

SIDE BRACES.—There shall be two side braces each 8"×12" framed at top and bottom as shown, and bolted at each end by five ¾" carriage bolts. They shall be joined together and to the loft timbers by six timbers shaped as shown. These timbers shall be bolted at each end and crossing by two ⅝" carriage bolts.

LOFT TIMBERS.—There shall be fourteen 4"×10" and twelve 4"×6" loft timbers arranged as shown. They shall be well fitted into gins, back braces, and side braces and fastened with two ⅝" carriage bolts at each end. In addition each pair of 4"×10" timbers shall be tied together at the gin ends by a ¾" rod with nuts on each end. A choking device with the necessary sheaves shall be fitted at the top loft.

DECKING.—Each loft shall be completely decked over, except for a space for hose sheave and counterweight, with 1¼"×8" material laid with one-inch spaces, well nailed with 16d. wire nails. The space between longitudinal sills from gins to boiler and a working platform around engine shall be decked with 2" material properly supported and fastened with 20d. wire nails.

DIAGONAL BRACES.—The diagonal braces shall be 4"×10" and 6"×8" arranged as shown. They shall be well fitted to gins and braces and secured at each end by two ⅝" carriage bolts.

HEAD BLOCK.—The arrangement of the head block timbers, sheaves and bearings shall be as shown in detail in Fig. 48. The timbers shall be well fitted and securely bolted. The sheaves shall be cast steel with turned groove and pins. The pile sheave pin shall have a hole drilled in each end and connect with each sheave bearing, and be fitted with two compression grease cups, Lukenheimer Ideal No. 3, or equal. This sheave pin shall be held in place by a taper pin in each bearing. The hammer line sheave shall have the pin pressed in and pinned. The bearings shall be lined with genuine babbitt, bored true, channeled for oil and each shall be fitted with an automatic grease cup, Lukenheimer Ideal No. 3, or equal.

HAMMER AND ROPE.—The hammer shall be cast iron, weighing 3800 pounds, and shall be in accordance with detail drawing furnished. The hammer line and pile lines will be furnished by the United States.

HOSE SHEAVE AND FITTINGS.—The hose sheave, holder, guides, etc., shall be made complete as shown in detail and assembled as shown. The sheaves shall be cast iron and have turned grooves and pins, all fitted up in first-class

manner. Oil holes shall be provided where required. The cast-iron counterweight shall be made in sections and shall be at least 100 pounds heavier than the assembled sheave holder, sheave, and hose full of water. It shall connect to the sheave holder by a ½" pliable steel cable. 65' of 2½" double-jacket rubber-lined fire hose shall be furnished and connected to the water piping and both ends shall have hose couplings.

Oil Tanks.—There shall be two oil tanks, each 2' 8" in diameter by 16' long, built of $\frac{3}{16}$" tank steel with bumped heads. They shall be tested for tightness by a hydrostatic pressure of 20 pounds. They shall be supported in saddles as shown and be held down by lugs, and screw bolts. Each tank shall have pressed steel flanges for filling pipe, sounding pipe, suction pipe, and drain plug.

Air Receiver.—The air receiver shall be 2' 6" in diameter by 10' long, supported in saddles and held down by lugs and screw bolts in the location shown. The shell shall be $\frac{5}{16}$" flange steel and the bumped heads ⅜", well riveted as required. It shall be tested by a hydrostatic pressure of 225 pounds per square inch. Pressed steel flanges shall be provided for inlet, outlet and drain plug.

Engine.—The engine shall be an American Hoist & Derrick Company, or equal, 8¼"×10" double-cylinder engine with two drums and four clutch winch heads. It shall be built for a working steam pressure of 125 pounds per square inch. Each drum shall be 14" diameter and 27" long between flanges. The upper drum shall be lagged to a diameter of 18" for the hammer line. The width between foundation bolts shall be approximately 4'. The engine shall be complete with throttle valve, lubricator, and efficient means for lubricating all bearings.

Steam Capstan.—A single-barrel steam capstan, American Ship Windlass Company, or equal, shall be installed in the location shown. Each steam cylinder shall be 5"×7" and the barrel 10½" in diameter. Additional timbers shall be installed as found necessary in order to securely fasten it in place.

Boiler.—The boiler shall be 40 h.p., portable locomotive type, with steam dome, water bottom and water front. It shall be built for a working pressure of 125 pounds per square inch and subjected to the Hartford Boiler Insurance Company inspection under hydrostatic pressure of 190 pounds. The boiler shall be about 16' long and 42" diameter of shell. The design and make shall be approved by the contracting officer before installation.

The boiler shall be equipped with the following fittings: smoke stack built of 16 gage steel, with steel hood and housing at upper deck and four guy wires; two sets of grate bars for burning wood; a Glafke, or equal, oil burner with heater; a 1" Metropolitan, or equal, automatic injector; Crosby pop safety valve; 3" chime bell whistle, 3 gage cocks into shell; water column and gage; steam gage with syphon and cock; 2 feet valves; 2 feed check valves; 1 blowoff cock.

Pumps.—There shall be a 3"×2"×3" duplex-boiler feed pump Worthington, or equal. The pump shall have Tobin bronze piston rods, composition-lined cylinders and composition valves. It shall be properly supported as directed and be connected as specified under piping.

The jet pump shall be a 10"×6"×10" Worthington, or equal, outside center-packed plunger pump having Tobin bronze piston rods, and composition plungers with composition bushed plunger stuffing boxes. The steam pipe shall be fitted with a suitable sight-feed lubricator of approved design and the discharge shall have a 6" dial pressure gage. This pump shall be supported on 12"×12" timbers and be securely fastened to them. The pipe connections shall be as specified under piping.

Air Compressor.—A Westinghouse, or equal, standard 11″ steam-driven, water-cooled, air compressor shall be installed in the location shown, supported by two 4″×6″ timbers framed into the house coaming and plate. The compressor shall be complete with governor, 4″ pressure gage, and sight-feed lubricator.

A 50-gallon galvanized-iron cooling tank shall be installed near the compressor and connected to it and to the feed pump discharge line.

Air Piping.—The air compressor shall be connected to the receiver by 1¼″ pipe with a 1¼″ globe valve and a pop safety valve in a convenient location. A 1¼″ globe valve shall be placed at the receiver outlet from which a 1¼″ pipe shall lead to the back braces where it shall pass through the deck and connect with two ¾″ globe valves and a 1″ pipe. This pipe shall lead to the fourth loft, with a plugged "T" at each loft, and terminate in two ¾″ globe valves.

Oil Piping.—Each oil tank shall have a 1½″ combined sounding and vent pipe placed as directed. They shall have 2½″ filling pipes leading into a "T," connecting with a 3″ filling stand in the deck, which shall have a composition cover and hose connection. Each tank shall have a 1½″ suction pipe leading in from the top and terminating in a 1½″ gate valve near the boiler; from these valves connection shall be made to the burner.

Water Piping.—The sea chest will be of steel pipe, furnished by the United States. It shall be secured in place by eight countersunk head bolts through 8″ filling timbers, placed as directed, and the hull shall be made water-tight around it. All water piping shall be galvanized.

The jet-pump suction shall have four separate suctions, made independent by using three 5″ flanged cross valves, one 5″ screwed angle valve and one 5″ flanged "T," made up with 5″ pipe as directed. One suction shall have a 5″ foot valve and strainer and shall lead into the sea chest. The other three shall lead one into each bilge compartment. The discharge shall be in two branches, a 4″ branch with angle valve leading into sea chest, and a 3″ branch leading aft as shown, with two 3″ gate valves placed as directed. The valves, fittings, etc., in the discharge pipe line shall be suitable for a working pressure of 175 pounds per square inch.

The feed pump and injector shall each have a 1½″ suction, with foot valve and strainer, from the sea chest and an independent discharge through feed heater to boiler. A Harrisburg No. 10, or equal, copper-coil feed-water heater shall be installed in the location shown. It shall be connected so as to use exhaust steam for heating. The drain shall be led overboard.

Steam Piping.—The steam pipe to the engine shall be 2½″ standard black pipe with a 2½″ globe valve near the boiler. The pipe to the jet pump shall be 2″ with a 2″ globe valve near the pump. The feed pump shall have a ½″ globe valve and be connected to the same pipe as the injector. A continuation of this pipe shall lead to the steam capstan which shall have a globe valve with stem extended through the deck in a convenient location. Three bilge syphons shall be installed, one in each compartment aft, discharging overboard.

The engine exhaust shall be 3″ black iron pipe from the engine to a 3″×4″×3″ "T" connecting with the jet-jump exhaust, which shall also be 3″ pipe, and shall be led under false deck between gin sills. From the "T" a 4″ black iron pipe shall lead to the feed-water heater and from the heater it shall join the safety valve exhaust and project 3′ above upper deck. It shall be properly flashed where passing through upper deck. The feed pump and steam capstan exhausts shall lead into the 4″ pipe before it enters the heater. All steam piping shall

be tested by a hydrostatic pressure of 190 pounds per square inch. Where practicable long-radius fittings shall be used in steam piping.

After the boiler has been painted it shall be covered with asbestos and magnesia bricks $1\frac{1}{2}''$ thick wired in place and crevices smoothly troweled with plastic magnesia covering. The surface shall be smoothly glazed and canvased in an approved manner. After testing, steam pipes, steam cylinders, and heater shall be covered with sectional magnesia pipe covering jacketed with canvas and secured with brass bands.

CLEATS.—There shall be four 42″ cast-iron cleats located as directed. Each cleat shall be set on a $12''\times36''\times\frac{1}{2}''$ steel plate, and shall be secured by two 1″ bolts passing through plate, deck, and an 8″ channel-iron bearing under two deck beams.

CHOCKS.—There shall be four roller chocks located as shown. Each sheave shall be 6″ diameter held between two plates, the lower one 12″ wide with outboard edge turned over edge of timber; the top plate shall be 8″ wide. Both plates shall be well bolted together and into sills and transoms.

GYPSEYS.—There shall be one ratchet gypsey windlass, "Providence" size C, or equal, well secured in the location shown. The length of shaft shall be as shown. There shall be two "Providence" size C, or equal, ratchet gypsey half windlasses, well fastened in the locations shown.

SALT POCKETS AND SALT.—Salt pockets shall be constructed on the inside of each gunwale by nailing $1''\times8''$ material to the clamp stringers between the deck beams. Pockets shall be formed on each side of each solid bulkhead by beveling the edges of $2''\times8''$ material so that it stands at an angle of about 30 degrees from the bulkhead, being nailed on the top edge to the under side of deck beams and on the bottom edge to the bulkhead. All salt pockets shall be filled with half-ground rock salt.

PAINTING.—All joints in gunwales, bulkheads and transoms shall be coated with boiled linseed oil applied at or about boiling temperature just before they are made up. All timber ends shall receive a coat of boiled linseed oil at boiling temperature before being covered, the posts and cross bracing ends in trussed stringers being dipped in boiling oil.

The entire bottom, sides and ends up to a 22″ water line shall be painted with two coats of Woolsey's or equal, copper paint, the last coat applied just before launching. The entire hull, except deck, above the water line, house, etc., shall receive three coats of pure white lead and boiled linseed oil, the last coat tinted to a light lead color.

The boiler, unfinished parts of engine, pumps, steam capstan, piping and all other iron work shall receive two coats of red lead and boiled linseed oil, the second coat tinted as directed. The smoke stack shall receive two coats of asphaltum paint.

APPENDIX VIII

UNITED STATES GOVERNMENT SPECIFICATIONS 59C1 FOR PORTLAND CEMENT *

DEFINITION

(1) The cement shall be the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate mixture of properly proportioned argillaceous and calcareous substances, with only such additions subsequent to calcination as may be necessary to control certain properties. Such additions shall not exceed 3 per cent by weight of the calcined product.

COMPOSITION

(2) In the finished cement, the following limits shall not be exceeded:

	Per Cent.
Loss on ignition for 15 minutes	4
Insoluble residue	1
Sulphuric anhydride (SO_3)	1.75
Magnesia (MgO)	4

SPECIFIC GRAVITY

(3) The specific gravity of the cement shall be not less than 3.10. Should the cement as received fall below this requirement, a second test may be made upon a sample heated for thirty minutes at a very dull red heat.

FINENESS

(4) Ninety-two per cent. of the cement, by weight, shall pass through the No. 100 sieve, and 75 per cent. shall pass through the No. 200 sieve.

SOUNDNESS

(5) Pats of neat cement prepared and treated as hereinafter prescribed shall remain firm and hard, and show no sign of distortion, checking, cracking, or disintegrating. If the cement fails to meet the prescribed steaming test, the

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cement may be rejected or the steaming test repeated after 7 or more days, at the option of the engineer.

Time of Setting

(6) The cement shall not acquire its initial set in less than forty-five minutes, and must have aquired its final set within ten hours.

Tensile Stength

(7) Briquettes made of neat cement, after being kept in moist air for twenty-four hours and the rest of the time in water, shall develop tensile strength per square inch as follows:

	Pounds.
After 7 days	500
After 28 days	600

(8) Briquettes made of 1 part cement and 3 parts standard Ottawa sand, by weight, shall develop tensile strength per square inch as follows:

	Pounds.
After 7 days	200
After 28 days	275

(9) The average of the tensile strengths developed at each age by the briquettes in any set made from one sample is to be considered the strength of the sample at that age, excluding any results that are manifestly faulty.

(10) The average strength of the sand mortar briquettes at twenty-eight days shall show an increase over the average strength at seven days.

Brand

(11) Bids for furnishing cement or for doing work in which cement is to be used shall state the brand of cement proposed to be furnished and the mill at which made. The right is reserved to reject any cement which has not established itself as a high-grade Portland cement, and has not been made by the same mill for two years and given satisfaction in use for at least one year under climatic and other conditions at least equal in severity to those of the work proposed.

Packages

(12) The cement shall be delivered in sacks, barrels, or other suitable packages (to be specified by the engineer), and shall be dry and free from lumps. Each package shall be plainly labeled with the name of the brand and of the manufacturer.

(13) A sack of cement shall contain 94 pounds, net. A barrel shall contain 376 pounds, net. Any package that is short weight or broken or that contains damaged cement may be rejected, or accepted as a fractional package at the option of the engineer.

Inspection

(14) The cement shall be tested in accordance with the standard methods hereinafter prescribed. In general the cement will be inspected and tested after

delivery, but partial or complete inspection at the mill may be called for in the specifications or contract. Tests may be made to determine the chemical composition, specific gravity, fineness, soundness, time of setting, and tensile strength, and a cement may be rejected in case it fails to meet any of the specified requirements. An agent of the contractor may be present at the making of the tests or they may be repeated in his presence.

(15) In case of the failure of any of the tests, and if the contractor so desires, the engineer may, if he deems it to the interest of the United States, have any or all of the tests made or repeated by the Bureau of Standards, United States Department of Commerce and Labor, in the manner hereinafter specified, all expenses of such tests to be paid by the contractor. All such tests shall be made on samples furnished by the engineer.

Standard Methods of Testing—Sampling

(16) The selection of the samples for testing will be left to the engineer. The number of packages sampled and the quantity to be taken from each package will depend on the importance of the work, the number of tests to be made, and the facilities for making them.

(17) The samples should be so taken as to represent fairly the material, and, where conditions permit, at least 1 barrel in every 50 should be sampled. Before tests are made, samples shall be passed through a sieve having 20 meshes per linear inch to remove foreign material. Samples shall be tested separately for physical qualities, but for chemical analysis mixed samples may be used. Every sample should be tested for soundness, but the number of tests for other qualities will be left to the discretion of the engineer.

Chemical Analysis

(18) The method to be followed for the analysis of cement shall be that proposed by the committee on uniformity in the analysis of materials for the Portland cement industry, reported in the *Journal of the Society for Chemical Industry* (Vol. 21, p. 12, 1902), and published in *Engineering News* (Vol. 50, p. 60, 1903), and in the *Engineering Record* (Vol. 48, p. 49, 1903).

(19) The insoluble residue shall be determined on a 1-gram sample which is digested on the steam bath in hydrochloric acid of approximately 1.035 specific gravity until the cement is dissolved. The residue is filtered, washed with hot water, and the filter paper and contents digested on the steam bath in a 5 per cent. solution of sodium carbonate. The residue is then filtered, washed with hot water, then with hot hydrochloric acid approximately of 1.035 specific gravity, and finally with hot water, then ignited and weighed. The quantity so obtained is the insoluble residue.

Determination of Specific Gravity

(20) The determination of specific gravity may be made with a standardized apparatus of Le Chatelier or other equally accurate form. Benzine (62° Baumé naphtha), or kerosene free from water, should be used in making the determination. The cement should be allowed to pass slowly into the liquid of the volumenometer, taking care that the powder does not adhere to the sides of the graduated

tube above the liquid, and that the funnel through which it is introduced does not touch the liquid. The temperature of the liquid in the flask should not vary more than 1° F. during the operation. To this end the flask should be immersed in water. The results of repeated tests should agree within 0.01.

(21) If the specific gravity of the cement as received is less than 3.10, a redetermination may be made as follows: Seventy grams of the cement are placed in a nickel or platinum crucible about 2 inches in diameter and heated for thirty minutes at a temperature between 419° C. and 630° C. After the cement has cooled to atmospheric temperature the specific gravity shall be determined in the same manner as described above. The cement should be heated in a muffle or other suitable furnace, the temperature of which is to be maintained above the melting-point of zinc (419° C.), but below the melting-point of antimony (630° C.). This maximum temperature can be recognized as a very dull red which is just discernible in the dark.

Determination of Fineness

(22) The Nos. 100 and 200 sieves shall conform to the standard sieve specifications of the Bureau of Standards, Department of Commerce and Labor.

(23) The determination of fineness should be made on a 50-gram sample which may be dried at a temperature of 100° C. (212° F.) prior to sifting. The coarsely screened sample should be weighed and placed on the No. 200 sieve, which, with the pan and cover attached, should be held in one hand in a slightly inclined position, and moved forward and backward in the plane of inclination, at the same time striking the side gently about 200 times per minute against the palm of the other hand on the upstroke. The operation is to be continued until not more than 0.05 gram will pass through in one minute. The residue should be weighed, then placed on the No. 100 sieve and the operation repeated. The sieves should be thoroughly clean and dry. Determination of fineness may be made by washing the cement through the sieve or by a mechanical sifting device which has been previously standardized with the results obtained by hand sifting on equivalent samples. In case of the failure of the cement to pass the fineness requirements by the washing method or the mechanical device, it shall be tested by hand.

Mixing Cement Pastes and Mortars

(24) The quantity of cement or cement and sand to be used in the paste or mortar should be expressed in grams and the quantity of water in cubic centimeters. The material should be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand be used, and a crater formed in the center, into which the proper percentage of clean water should be poured; the material on the outer edge should be turned into the crater by the aid of a trowel. As soon as the water has been absorbed, the operation should be completed by vigorously mixing with the hands for one minute and a half. During the operation of mixing, the hands should be protected by rubber gloves. The temperature of the room and the mixing water should be maintained as nearly as practicable at 21° C. (70° F.).

Determination of Normal Consistency

(25) The normal consistency for neat paste to be used in making briquettes and pats should be determined by the ball method, as follows:

(26) A quantity of cement paste should be mixed in the manner above described under Mixing Cement Pastes and Mortars, and quickly formed into a ball about 2 inches in diameter. The ball should then be dropped upon a hard, smooth, and flat surface from a height of 2 feet. The paste is of normal consistency when the ball does not crack and does not flatten more than one-half of its original diameter.

(27) Trial pastes should be made with varying percentages of water until the correct consistency is obtained.

(28) The percentage of water to be used in mixing mortars for sand briquettes is given by the formula:

$$y=\frac{2}{3}\frac{P}{n+1}+K,$$

in which y is the percentage of water required for the sand mortar; P is the percentage of water required for neat cement paste of normal consistency; n is the number of parts of sand to one of cement by weight; and K is a constant which for standard Ottawa sand has the value 6.5.

The percentage of water to be used for mortars containing three parts standard Ottawa sand by weight to one of cement is indicated in the following table:

Percentage of Water for Neat Cement Paste.	Percentage of Water for 1 to 3 Mortars of Standard Ottawa Sand.
18	9.5
19	9.7
20	9.8
21	10.0
22	10.2
23	10.3
24	10.5
25	10.7
26	10.8
27	11.0
28	11.2
29	11.3

Determination of Soundness

(29) Pats made of neat cement paste of normal consistency about 3 inches in diameter, ½ inch in thickness at the center, and tapering to a thin edge, should be kept in moist air for a period of twenty-four hours. One pat should then be kept in air and a second in water, at the ordinary temperature of the laboratory, not to vary greatly from 21° C. (70° F.), and both observed at intervals for at

least 28 days. A third pat should be exposed to steam at atmospheric pressure above boiling water for five hours.

Determination of Time of Setting

(30) The time of setting should be determined by the standardized Gilmore needles, as follows:

A pat of neat cement paste about 3 inches in diameter and $\frac{1}{2}$ inch in thickness with flat top mixed at normal consistency should be kept in moist air, at a temperature maintained as nearly as practicable at 21° C. (70° F.). The cement is considered to have acquired its initial set when the pat will bear, without appreciable indentation, a needle $\frac{1}{10}$ inch in diameter loaded to weigh $\frac{1}{4}$ pound. The final set has been acquired when the pat will bear without appreciable indentation, a needle $\frac{1}{24}$ inch in diameter, loaded to weigh 1 pound. In making the test the needle should be held in a vertical position and applied lightly to the surface of the pat. The pats made for soundness test may be used to determine the time of setting.

Tensile Tests

(31) Tensile tests should be made on an approved machine. The test pieces shall be briquettes of the form recommended by the committee on uniform tests of cement of the American Society of Civil Engineers, and illustrated in Circular 33 of the Bureau of Standards. The briquettes shall be made of paste or mortar of normal consistency. Immediately after mixing, the paste or mortar should be placed in the molds, pressed in firmly by the fingers, and smoothed off with a trowel without mechanical ramming. The material should be heaped above the mold, and in smoothing off, the trowel should be drawn over the mold in such a manner as to exert a moderate pressure on the material. The molds should be turned over and the operation of heaping and smoothing repeated. Not less than three briquettes should be made and tested for each sample for each period of test. The neat tests are not considered so important as the sand tests. The briquettes should be broken as soon as they are removed from the water. The load should be applied at the rate of 600 pounds per minute.

Storage of Test Pieces

(32) During the first twenty-four hours after molding the test pieces should be kept in air sufficiently moist to prevent them from drying. After twenty-four hours in moist air the test pieces should be immersed in water. The air and water should be maintained as nearly as practicable at 21° C. (70° F.).

Standard Sand

(33) The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve.

(34) Sand having passed the No. 20 sieve shall be considered standard when not more than 2 grams pass the No. 30 sieve after one minute continuous sifting of a 200-gram sample.

(35) The No. 20 and No. 30 sieves shall conform to the standard sieve specifications of the Bureau of Standards, Department of Commerce and Labor.

Copies of the above specifications can be obtained upon application to the various navy pay officers or to the Bureau of Supplies and Accounts, Navy Department, Washington, D. C.

References: Y. and D., 5766, Nov, 20, 1903; Y. and D., 8789, June 6, 1905; Y. and D., 5766WM, Nov. 8, 1906; Y. and D., 5766, Nov. 22, 1907; Y. and D., 10015, Oct. 23, 1909; Navy Dept., 5471-77-1, Mar. 23, 1912; Y. and D., 10061-63-Ca.-AM, Mar. 25, 1912.

APPENDIX IX

SPECIFICATIONS (59C2) FOR CONCRETE AND CONCRETE MATERIALS AND MORTAR AND MORTAR MATERIALS *

The following specifications govern in the case of all contracts of which they form a part, except in so far as they may be modified by the special specifications for the work.

I. MATERIALS FOR CONCRETE AND MORTAR

(*a*) *Sand for concrete* shall be clean and siliceous, and shall be a well-graded mixture of coarse and fine grains, with the coarse grains predominating. It shall be free from clay, loam, mud, organic matter, or other impurities. It shall be screened to remove all particles not passing a ¼-inch mesh screen, unless, in the opinion of the officer in charge, the proportion of particles above ¼ inch is so small that the sand will perform its functions in the concrete without screening. Sand for concrete may contain not more than 5 per cent of silt when measured by volume by shaking a sample of the material with water in a test-tube and allowing it to subside. Crusher dust or screenings passing a ¼-inch mesh screen may be combined with and measured as sand, but not more than one-third of the sand in any one batch may be of this material, unless it can be shown that the sizes of the particles are practically in the same proportion as in the most suitable grades of natural sand. Sample of the sand may be submitted to the officer in charge for approval before bidding if desired.

(*b*) *Sand for mortar* shall be clean and siliceous, and shall be composed of grains of varying size. It shall be free from clay, loam, mud, salt, organic matter, or other impurities, and shall also be free from silt. It shall be screened, if necessary, to remove all particles not passing through a ¼-inch mesh screen. If joints in the brickwork are too thin to allow the use of particles of ¼-inch size, then the screen used shall be of such a mesh as to exclude particles not suitable for use in the particular thickness of joint in use.

(*c*) *Broken stone.*—Crushed granite, trap, gneiss, or other equally suitable rock, may be used for concrete. It shall be free from clay, loam, mud, organic matter, and other impurities. Fine crushed stone passing a ¼-inch mesh screen may be combined with and measured as sand for concrete, as specified under "Sand for Concrete."

(*d*) *Gravel.*—Screened gravel may be used in lieu of broken stone where the latter is specified. Gravel shall be composed of hard, durable stone, and shall be clean, free from slaty or soft stones, clay, loam, mud, organic matter, and other impurities.

* Issued by the Navy Department, March 25, 1912.

(*e*) *Sizes of broken stone and gravel.*—Materials shall be screened to size and shall be run of the crusher or of the bank between the limits given. The particles shall vary in size between the upper and lower limits in order that the voids may be a minimum. For foundations or mass concrete the stone shall pass a 2-inch screen and be retained on a ¼-inch screen; for reinforced concrete the stone shall pass a 1-inch screeen and be retained on a ¼-inch screen, but when the distance between the reinforcing strands is less than 2 inches the upper limit shall not be over ¾ inch.

(*f*) *Water.*—Only fresh and clean water shall be used in mixing concrete and mortar.

(*g*) *Cement* shall be in accordance with the United States Government specifications for Portland cement as issued by the Navy Department. Cement furnished as a part of a public works contract shall be stored by the contractor, immediately upon delivery, in a suitable weather-tight and properly ventilated place, having a floor raised above the ground. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

(*h*) *Lime* shall be of the best quality, fat, well burned, and perfectly fresh lump lime, of a brand well known to the trade or an established brand of hydrated lime. All lime shall be shipped to the site in barrels bearing the name and label of the brand.

II. MEASUREMENT OF MATERIALS

(*a*) *Cement* shall be measured by weight and not by volume, and for the purpose of proportioning concrete or mortar 100 pounds shall be taken as the equivalent of 1 cubic foot. A sufficient number of the bags or barrels shall be actually weighed to insure the required amount in each batch.

(*b*) *Sand, broken stone, or gravel* shall be measured by volume for each batch in boxes, barrels, or other equally effective measuring devices approved by the officer in charge.

III. PROPORTIONING MATERIALS IN CONCRETE

(Method A, fixed volumes of sand and stone.)

(*a*) *Mass concrete* (1–3–6).—Foundations for buildings, including wall foundations, column piers, curtain walls, retaining walls in earth, abutments, wall footings, concrete foundations, and mass concrete similar to these shall be composed of 1 part cement (allowing 100 pounds to the cubic foot), 3 parts by volume of sand, and 6 parts by volume of broken stone or gravel.

(*b*) *Reinforced concrete* (1–2–4) in columns, beams, slabs, walls, etc., shall be composed of 1 part cement (allowing 100 pounds to the cubic foot), 2 parts by volume of sand, and 4 parts by volume of broken stone or gravel.

(Method B, fixed volume of stone and variable volume of sand.)

(*c*) *Proportions of concrete* under Method B, which is to be used only when specially required by the specifications for the work, shall be as follows:

Class A.—One part cement (allowing 100 pounds to the cubic foot) to 6½ parts by volume of broken stone combined with a variable proportion of sand.

Class B.—One part cement (allowing 100 pounds to the cubic foot) to $4\frac{1}{2}$ parts by volume of broken stone combined with a variable proportion of sand.

(*d*) *Determining amount of sand.*—It is the intention with the given amounts of cement and broken stone to secure concrete as dense as possible. The amount of sand to be added will therefore depend on the actual character of the sand and broken stone, and the number of cubic feet of sand to be added will be determined by frequent experiments and tests of the material as actually delivered and accepted during the progress of the work. The proportion will be established from time to time for each class of aggregate used and will not be changed during any period of twenty-four hours, unless the contractor desires to use materials of different characteristics, and the proportion determined at any one time shall continue to be used until the next determination is made and the contractor has been ordered to change the previous proportions.

(*e*) *Tests for amount of sand* shall be conducted as follows: To a fixed volume (not less than 5 cubic feet) of dry stone, which shall be a representative sample, add a fixed volume of dry sand. Mix these very thoroughly, so as to fill all the voids as uniformly as possible, and then measure and weigh the resulting mixture. Repeat the experiment with varying amounts of sand. That proportion of sand which gives the heaviest mixture per unit of volume shall be used in preparing the concrete. The labor and materials for the experiments shall be furnished by the contractor, but the experiment shall be conducted under immediate supervision of the officer in charge.

(*f*) *Run of crusher stone.*—The specifications elsewhere require the screening out of particles of broken stone and gravel below $\frac{1}{4}$ inch in size. Under method " B " of proportioning materials, this will not be necessary if it can be shown by tests that the amount of fine material remaining in the stone or gravel is such that with the addition of sand the resulting total of fine material below $\frac{1}{4}$ inch size will be at least equal to the most suitable grades of natural sand. Tests must be made to determine that there is not an excess of fine material beyond that required to give the densest possible aggregate. Permission to use run of crusher stone as described in this paragraph must be previously obtained from the officer in charge of the work.

IV. MIXING MATERIALS

(*a*) *Method of mixing.*—Concrete shall be mixed by hand or by a mechanical mixer of a type to be approved by the officer in charge.

(*b*) *Presence of inspector.*—Concrete may be mixed and placed only in the presence of an inspector, and the contractor or his agent shall give due and ample notice to the officer in charge when mixing is to be commenced. The officer in charge may reject any concrete mixed or placed without the presence of an inspector when such notice has not been given.

(*c*) *Hand mixing.*—For mixing by hand use only strong, water-tight, well-built platforms, large enough to provide space for the partial simultaneous mixing of two batches, which shall not consist of more than 1 cubic yard each, shall be used. The sand and cement in specified proportions shall be spread in layers on the mixing platform, the sand at the bottom, and shall then be thoroughly and evenly mixed dry until of uniform color without streaks. Water shall then be added and thoroughly and uniformly incorporated. The sand and cement mixture shall then be placed on the broken stone, which shall have been pre-

viously wet down and placed in a layer on the board adjoining the cement and sand. The mixture shall then be worked up and turned over at least three times, not including shoveling from the platform to place of deposit or into the vehicle transportation. The material on the shovel shall be completely turned over and not merely dropped from the shovel. The number of turnings shall be more than three if necessary to produce a thoroughly mixed concrete of uniform consistency throughout. The use of a rake is permitted in mixing the sand and cement, but is forbidden after the stone has been added. The amount of water shall be such as to give the consistency specified elsewhere, and special care must be taken not to exceed the proper amount. Should an excess of water be added inadvertently, the batch shall be remixed with sufficient additional cement to take up the water. The details of the method of hand mixing may be varied from the above by permission of the officer in charge, but the results must be substantially the same as produced by the method described.

(*d*) *Machine mixing*, if efficiently done, is ordinarily to be preferred to hand mixing. The mixer shall be tested at the beginning of the work to determine the most efficient size of batch and method of use. The materials for each batch shall be carefully and accurately measured and introduced into the mixer. Water shall be introduced into the mixer during the process of mixing, and the amount shall be accurately measured to give the required consistency as determined by experience with previous batches. Should too much water be added inadvertently, the batch shall be remixed with sufficient additional cement to take up the water. The machine shall be of the batch type, and shall produce a concrete thoroughly mixed and of uniform mixture and consistency superior to or at least equal to that produced by the hand mixing above described. The type of machine selected is subject to approval by the officer in charge. A machine of the continuous or of the gravity type shall not be used unless specially permitted by the specifications for the work.

(*e*) *Consistency.*—A medium or quaking mixture of tenacious jelly-like consistency which quakes under light ramming shall be used for all concrete except reinforced concrete. For reinforced concrete a very wet or mushy mixture shall be used, such that it will flow into the forms and around the reinforcement, but not so wet as to allow the materials to separate in handling or transporting. For reinforced roof concrete, the mixture shall be slightly dryer.

V. HANDLING AND PLACING CONCRETE

(*a*) *Handling.*—Concrete shall be conveyed and deposited in such a manner that there may be no separation of the different ingredients. Any concrete which has commenced to set before placing shall be rejected and immediately removed from the work and wasted in such manner as may be directed by the officer in charge.

(*b*) *Placing.*—The specified consistency of the concrete will require only light tamping or spading. Along the face of the forms the concrete shall be spaded in such a manner as to force back the larger particles and bring the mortar to the surface of the form in order to avoid pits and irregular concrete after removal of the forms. Except in continuous laying, the concrete shall not be deposited upon that which has been laid less than twelve hours. The surface of each layer which is not thoroughly clean or which has been in place more than twelve

hours shall be thoroughly cleaned and wet, and then broomed with one-to-one cement before depositing the succeeding layer.

(*c*) *Joints in reinforced concrete.*—All slabs shall be jointed over the centers of beams and girders. Beams and girders shall be built in sections continuously from the center line of one girder or column to the next, the deeper girder being recessed to receive the shallower one unless both are built simultaneously. Where both beams and girders intersect over the same column, the joints shall run diagonally across the column so as to secure proper bearing for each.

(*d*) *Protection of concrete after depositing.*—Concrete shall be protected from injurious action by the sun, heavy rains, currents of water, frost, or mechanical injury. In dry weather it shall be wet down frequently.

(*e*) *Freezing weather.*—Concrete shall not be deposited in freezing weather nor at such times as it is likely in the opinion of the officer in charge to be subjected to freezing weather within twenty-four hours after being deposited, except on the written authority of the officer in charge, and then only under such restrictions and in such manner, as he may prescribe.

(*f*) *Depositing under water.*—Concrete shall not be deposited under water unless distinctly permitted by the specifications for the work. Concrete shall at all times be protected from running water.

(*g*) *Forms.*—All forms shall be waterproof to prevent leakage of cement and of a substantial character, well braced, and sufficiently strong to hold the face of the concrete true to line. The design and character of forms shall be subject to approval by the officer in charge, and if not approved by him shall be remodeled and improved. If the forms are held together by bolts or wires, they shall be so made that no iron will be left exposed on the faces of the finished work. All forms for exposed work shall be surfaced one side and free from defects affecting the finished appearance of the concrete. Forms shall remain in place long enough to allow the concrete to properly set, as determined by the officer in charge.

VI. MORTARS

(*a*) *Cement mortar* shall be composed of 1 part Portland cement and 2 parts sand. The sand and cement in specified proportions shall be spread in layers on a closed-in platform or tight mortar box, the sand at the bottom, and then thoroughly and evenly mixed dry until of uniform color without streaks. Water shall then be added and thoroughly and uniformly incorporated with the mixture until it is of the proper consistency for use with the work for which it is intended.

(*b*) *Lime paste* shall be prepared in a tight box by adding to the lime sufficient water to thoroughly slack the lime without "burning" or "chilling." The slacked lime while still warm shall be sifted or run through a sieve having meshes $\frac{1}{8}$ inch square in order to remove all particles of unslacked or partially slacked lime. After cooling it shall be stored and properly protected. Lime paste for mortar shall stand one week before using.

(*c*) *Lime mortar* shall be composed of 2 parts of cement (allowing 100 pounds to the cubic foot), 1 part of lime paste by volume, and 6 parts of sand by volume. The lime paste and sand shall be thoroughly mixed not less than twenty-four hours before using, and 6 parts of the mixture shall be taken to mix with the 2 parts of cement. The cement shall be added only in small quantities sufficient for immediate use. Retempering of mortar in which cement has already begun

to set will not be permitted. The mortar must be used so that it will be in place within the limit of time of the initial setting of the cement.

Copies of the above specifications can be obtained upon application to the various navy pay offices or to the Bureau of Supplies and Accounts, Navy Department, Washington, D. C.

References: Y. and D., 9113-S-T, Mar. 1, 1912; Dept., 5471-88, Mar. 2, 1912; S. and A., 118159.

APPENDIX X

SPECIFICATIONS FOR CREOSOTING PILING (WATERSOAKED) *

THE piling to be treated must be Douglas fir, thoroughly sound and free from all defects calculated to impair its value; they must be perfectly straight from end to end, free from bark, Teredo, Limnoria or other seaworm holes, also from barnacles and similar attachments. They must be cut from mature stock and show an even taper from butt to point. The butts shall not be less than a full fourteen (14″) inches in diameter, nor less than nine (9″) inches in diameter at the point.

Each cylinder charge must be made up of piles which have been in the water as nearly as possible the same length of time; nor must they have been so long therein as to cause deterioration or damage of any kind.

After the piles are placed in the cylinder they must be immersed in creosote oil of the quality specified below, of a temperature ranging between 160 and 170° F., and kept covered during the entire boiling period under at least 4 inches of oil at the shallowest place. The engineer on duty must from time to time during the boiling satisfy himself by bleeding the cylinder that such is the case.

After filling the cylinder with oil, steam must then be regulated through the heating coils, so that the temperature within the cylinder is kept gradually rising as fast as the condensation will permit until 220° F. is reached; after which the steam pressure must only be such as to maintain a regular and constant temperature within the cylinder with 220° as the minimum and 225° F. the maximum until such time as the amount of condensation per cubic foot per hour collecting in the hot well of the condenser shows the interior of the wood to be thoroughly dry, when the steam pressure in the coils should be released, and the cylinder filled up with creosote oil from the storage or working tanks, of a temperature ranging between 160 and 170° F., then pump pressure applied until the gage shows 5 pounds pressure in the cylinder (this to insure the fact of the cylinder being actually full), after which the connection between the storage tank and cylinder should be closed and the connection between measuring tank and cylinder opened. Additional pressure must then be applied until the piling has taken the proper amount of oil, forced in under such conditions as will insure its complete retention in the wood after drip is over, and figured at the weight of the dry oil per gallon at 100° F., the cylinder doors may then be opened provided the temperature within is reduced to below 200° F.

* F. D. Beal, Manager, St. Helens Creosoting Co., Portland, Ore.

After treatment, the piling must be free from all heat checks, water bursts and other defects due to inferior treatment which would impair usefulness or durability for the purposes intended. Piles when bored midway between the ends must show no moisture in the center, and the borings beyond the oil penetration must retain their natural elasticity and strength.

GREEN OR FRESHLY CUT AND SEASONED PILING

Green or freshly cut piles delivered at the treating plant on cars, or any which have not been lying in the water, must be treated in the manner prescribed.

Thoroughly seasoned piles must be treated in the manner prescribed.

No piling in these two classes must be mixed together and treated in the same charge, and none in either of these two classes should be treated which is not at the time free from rot, and in proper condition for use after treatment as far as splits or breaks are concerned; if any such is received from the mills it should not be treated unless the inspector directs it to be done.

After the piles are placed in the cylinder, it must be immersed in creosote oil of a temperature ranging between 160 and 170° F., and kept covered during the entire boiling or heating period under at least 4 inches of oil at the shallowest place; the engineer on duty must from time to time during the boiling satisfy himself by bleeding the cylinder that such is the case.

In the case of green or freshly cut piling, steam must thereafter be regulated through the heating coils so that the temperature within the cylinder is kept gradually rising as fast as the condensation will permit until 212° F., is reached, with 215° F. as the maximum; after which the steam pressure must only be such as to maintain a regular and constant temperature within the cylinder between these figures, until such time as the amount of condensation per cubic foot per hour collecting in the hot well of the condenser shows the interior of the wood to be thoroughly dry, when steam pressure in the coils should be released.

In the case of thoroughly seasoned piling, the temperature of the oil in the cylinder must be allowed to rise slowly and steadily until 190° F. is reached, with 192° F. as the maximum; and kept between these figures until such time (depending upon the dimensions) as the interior of the wood shall have become sufficiently warmed up to enable it to take the required amount of oil, when the steam pressure in the coils should be released.

The cylinder should then (in each case) be filled up with creosote oil from the storage or working tank, of a temperature ranging between 160 and 170° F., and pressure from the pump applied until the gage shows 5 pounds pressure in the cylinder (this to insure the fact of the cylinder being actually full), after which the connection between the storage tank and cylinder should be closed, and the connection between measuring tank and cylinder opened. Additional pressure must then be applied slowly and steadily until the piling has taken the proper amount of oil forced in under such conditions as will insure its complete retention in the wood after drip is over, and figured at the weight of the dry oil per gallon at 100° F., the cylinder doors may then be opened provided the temperature within is below 200° F.

After treatment the piling must be free from all heat checks, water bursts, and other defects due to inferior treatment, which would impair usefulness or durability for the purposes intended.

FIR SAWED MATERIAL

Seasoned, and green or freshly sawed material must not be mixed together and treated in the same charge, and none should be treated, which is not at the time free from rot, and in the proper condition for use after treatment so far as splits or breaks are concerned, if any such is received from the mills, it should not be treated unless the inspector directs it to be done.

Square timber must not be treated in the same charge with planking, nor ties with planking, and sufficient 1-inch strips must be placed between each tier, with from ½ to 1 inch space left between each piece, so that the oil can have free access to all surfaces.

After the material is placed in the cylinder, it must be immersed in creosote oil of a temperature ranging between 160 and 170° F., and kept covered during the entire boiling or heating period under at least 4 inches of oil at the shallowest place; the engineer on duty must from time to time during the boiling satisfy himself, by bleeding the cylinder, that such is the case.

In the case of green or freshly sawed material, steam must thereafter be regulated through the heating coils so that the temperature within the cylinder is kept gradually rising as fast as condensation will permit until 212° F. is reached, with 215° F. as the maximum, after which the steam pressure must only be such as to maintain a regular and constant temperature within the cylinder between these figures, until such time as the amount of condensation per cubic foot per hour collecting in the hot well of the condenser shows the interior of the wood to be thoroughly dry, when steam pressure in the coils should be released.

In the case of thoroughly seasoned timber, the temperature of the oil in the cylinder must be allowed to rise slowly and steadily until 190° F. is reached, with 192° F. as the maximum, and kept between these figures until such time (dependent upon the dimensions) as the interior of the wood shall have become sufficiently warmed up to enable it to take the required amount of oil, when the steam pressure in the coil should be released.

The cylinder should then (in each case) be filled up with creosote oil from the storage or working tank, of a temperature ranging between 160 and 170° F. and the pressure from the pump applied until the gage shows 5 pounds pressure in the cylinder (this to insure the fact that the cylinder is actually full), after which the connection between the storage tank and the cylinder should be closed, and the connection between the measuring tank and the cylinder opened. Additional pressure must then be applied slowly and steadily until the material has taken the proper amount of oil, forced in under such conditions as will insure the complete retention in the wood after drip is over, and figured at the weight of the dry oil per gallon at 100° F.; the cylinder doors may then be opened provided the temperature within is below 200° F.

After treatment the material must be free from all heat checks, water bursts, and other defects due to inferior treatment, which would impair usefulness or durability for the purposes intended.

CREOSOTE OIL

The oil used in treating shall be the best obtainable grade of coal-tar creosote; that is it must be a pure product of coal-tar distillation, and must be free from

admixture of oils, other tars, or substances foreign to pure coal-tar; it must be completely liquid at 38° C., and must be free from suspended matter; the specific gravity of the oil at 38° C., must be at least 1.03. Creosote oil, when distillation is carried on as laid down in the American Railway Engineering and Maintenance of Way Association, Vol. 9, page 708, shall give no distillate below 200° C. not more than 5 per cent. below 210° C., not more than 25 per cent. below 235° C. and the residue above 355° C. if it exceed 5 per cent. in quantity, must be soft. The oil shall not contain more than 3 per cent. of water.

GENERAL CLAUSES

All material shall be treated to the entire satisfaction of the purchaser's inspector, he being allowed full access at all times to the facilities used in the treatment while it is in progress. When recording thermometers are installed on treating plants the inspector shall have access to same in order to check temperatures during treatment. He shall also be furnished with facilities for taking daily inventories on creosote oil in order to ascertain the exact amount injected in the timber; but the fact of any inspector being at the plant shall not relieve the treating company's officials from the responsibility of seeing that the treatment of all material is properly and carefully done, with the agreed penetration of oil in each case, based on the contract amount.

Before each cylinder charge is disposed of, the depth and character of the penetration must be ascertained by boring one or more auger holes, after wood has cooled, in as many pieces of each class of material as may be necessary for the purpose, and such pieces as are not found to be fully treated in accordance herewith must be returned to the cylinder with a subsequent charge for further treatment without extra cost therefor; should more than 10 per cent. of the total number of pieces treated be found defective, the entire charge must be so returned. No material must be removed from the treating yard until all auger holes are tightly plugged with creosoted plugs.

The intent of these specifications is that the wood when it comes out of the cylinder and after all drip is over, shall contain the full weight of oil to the cubic foot, forced in at such pressure, and under such conditions as to enable the wood cells and fiber to retain it permanently.

The pressure gages and thermometers must be compared and tested at frequent intervals with standard test appliances kept on hand for that purpose, and all differences corrected.

Competent and experienced engineers shall be in charge of the treatment night and day, and required to make frequent examinations of the temperature during the boiling, especially when the maximum heat is being applied; the thermometers being located so that they will correctly reflect the heat conditions within the cylinders, and at the same time be convenient to get at.

In handling the material after treatment, sharp dog or cant hooks must not be used in any way whereby the full protection of the treatment is likely to be lessened; in the case of piling they must be used in the space within 2 feet from the large end and 6 feet from the point. Any material broken or otherwise damaged in treatment or by careless handling, while in the treating company's care and until delivered to the purchaser's destination as per contract, will be rejected, and the treating company must substitute new material therefor or the cost will be charged to the treating company.

Facilities for testing the grade of the oil to ascertain that it is in accordance with the above specifications shall be furnished by the treating company and all necessary facilities for testing the same shall be provided, and kept at the plant by the treating company.

The treatment under these specifications will require 12 pounds of creosote oil per cubic foot.

APPENDIX XI

TABLE LXIX.—COMPRESSORS FOR PNEUMATIC WORK

INGERSOLL-RAND STRAIGHT LINE COMPRESSORS. CLASS A1

Steam Pressure 80–120 Pounds

Size of Cylinders, Inches. Diam. Steam Cyl.	Diam. Air Cyl.	Stroke.	R.P.M.	Piston Displacement Cu.Ft. Per Min.	Air Pres. Designed.	Boiler H.P. Max. Air Pres.
12	16¼	14	155	496	15–35	73
14	18¼	18	120	628	25–50	83
14	20¼	18	120	770	15–35	83
14	22¼	18	120	924	10–25	80
16	20¼	18	120	770	25–50	103
16	22¼	18	120	924	15–35	99
18	22¼	24	94	973	25–50	125
18	24¼	24	94	1150	15–40	131
18	26¼	24	94	1345	10–30	128
20	24¼	24	94	1150	25–50	156
20	26¼	24	94	1345	10–40	149
22	26¼	24	94	1345	30–50	174

Size of Pipes, Inches. Steam Inlet.	Steam Exhaust.	Air Discharge.	Circulating Water.	Length Over-all.	Width Over-all.	Height above Foundation.	Weight.
3	4	5	¾	12–7	3–9	3–4	8,900
3½	4	5	¾	15–3	4–3	4–3	13,800
3½	4	7	¾	15–3	4–3	4–3	14,200
3½	4	8	¾	16–0	4–3	4–3	14,600
4	5	7	¾	15–7½	4–3	4–3	14,800
4	5	8	¾	16–0	4–3	4–3	15,200
5	5	5	1	19–0	5–3	5–3	23,000
5	5	6	1	19–0	5–3	5–3	23,500
5	5	9	1	19–0	5–3	5–3	24,000
5	6	9	1	19–0	5–3	5–3	24,000
5	6	6	1	19–0	5–3	5–3	24,500
5	6	9	1	19–0	5–6	5–3	25,000

The compressor plant should be arranged with boilers and compressors placed similarly to the pumping scow, Fig. 70. The medical lock can be placed on the same scow. The compressor used on the Chillicothe work was about the size of the second one in the table, while the one used on the Vancouver work was the same as No. 10 in the table. It is always advisable to have a *duplicate* compressor in case of more air being needed in an emergency, or in case one machine is broken down.

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